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INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

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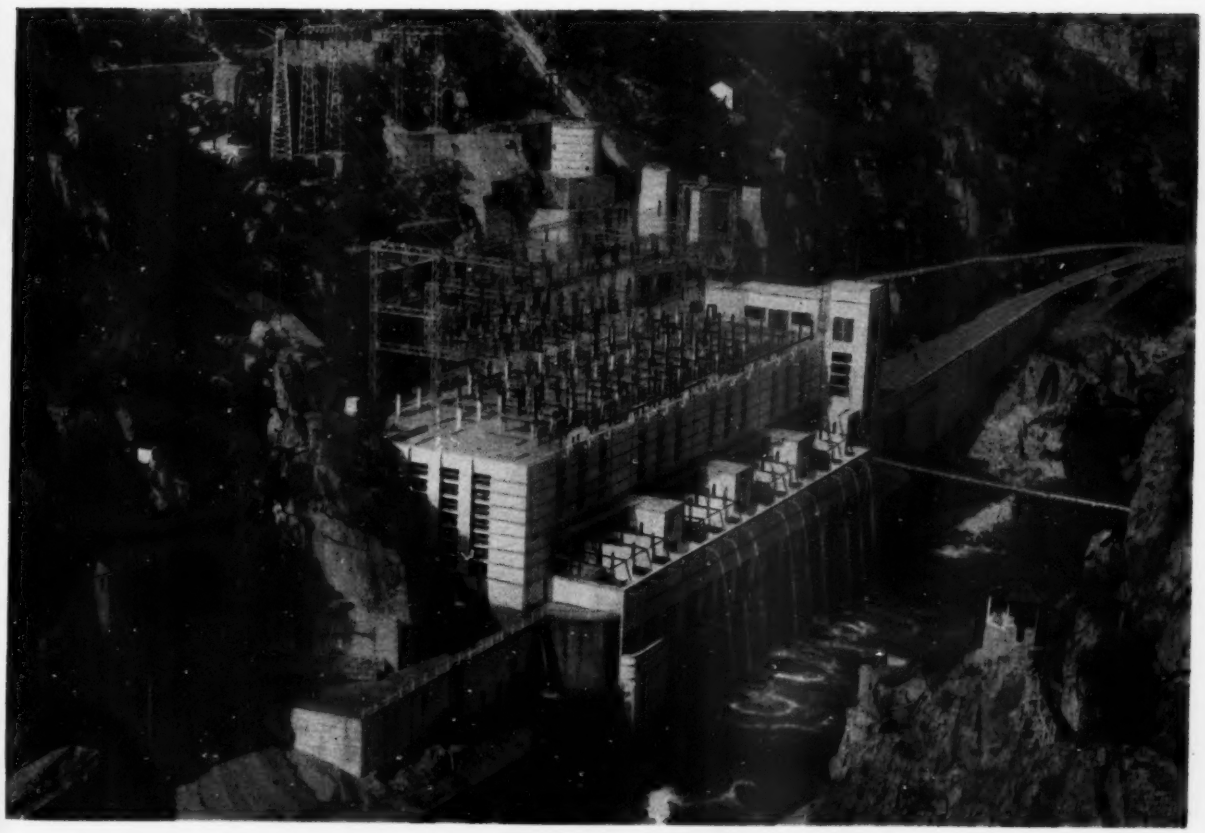
is the largest power station in Spain. This hydro-electric station is equipped with four 75,000 kVA a.c. generators, together with transformers and switchgear, supplied by Metropolitan-Vickers.



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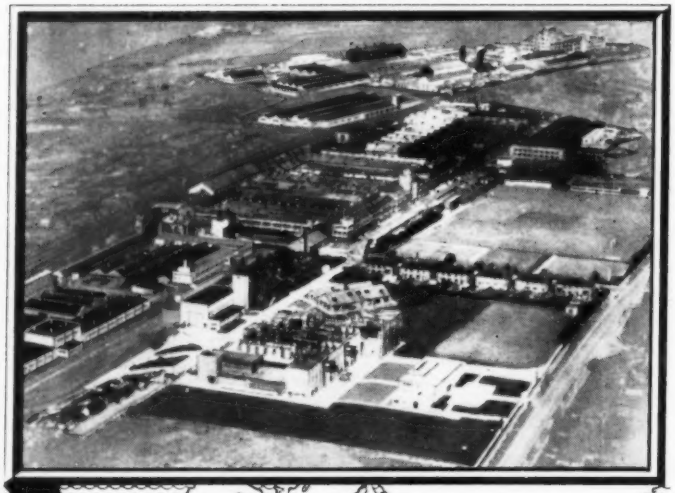

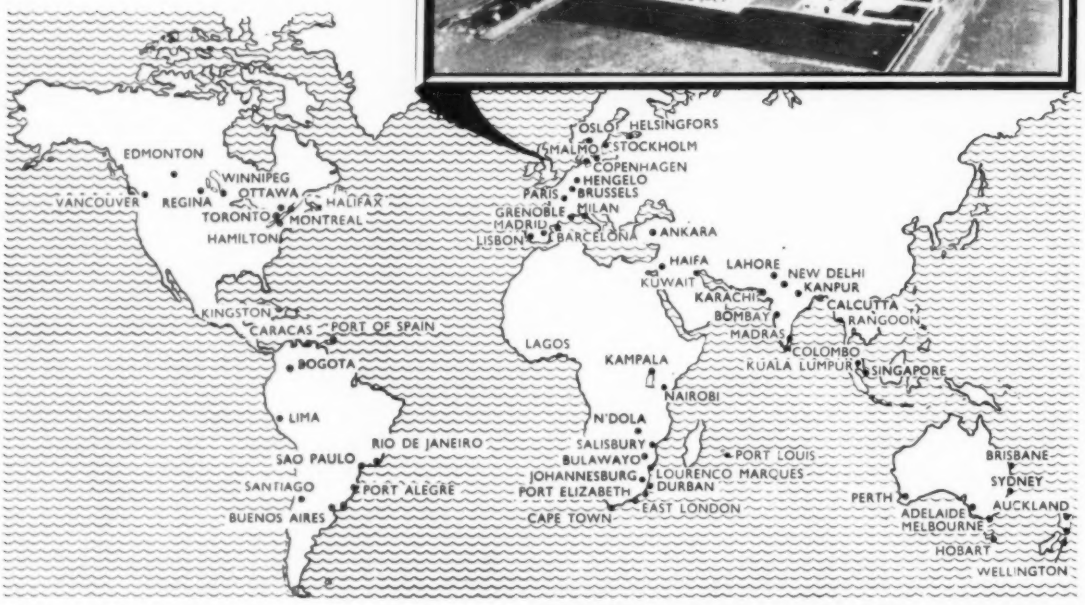

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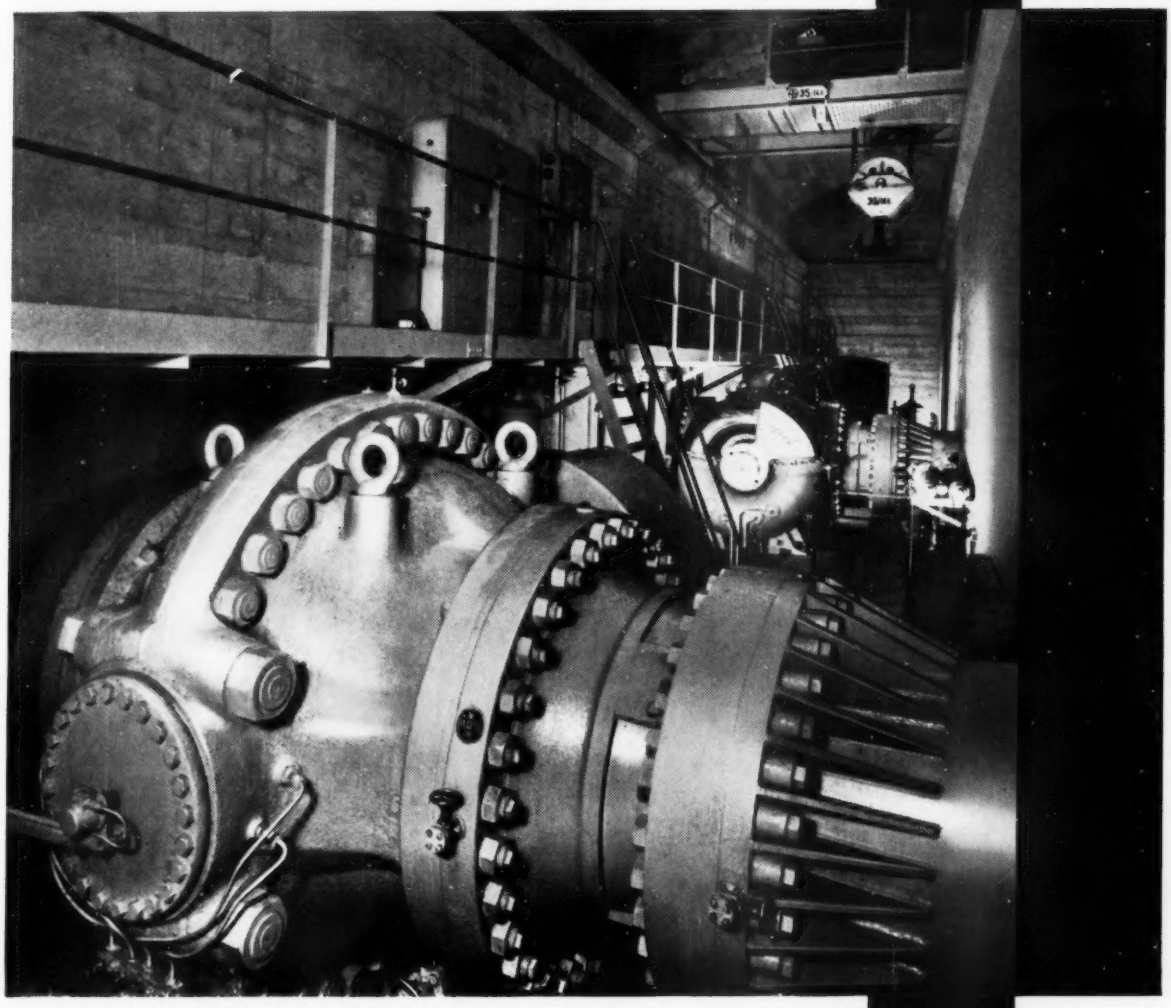
Equipment for Hydro-Electric Power Stations



Valve chamber with rotary slide sphériques valves as turbine shut-off device in a swiss power station

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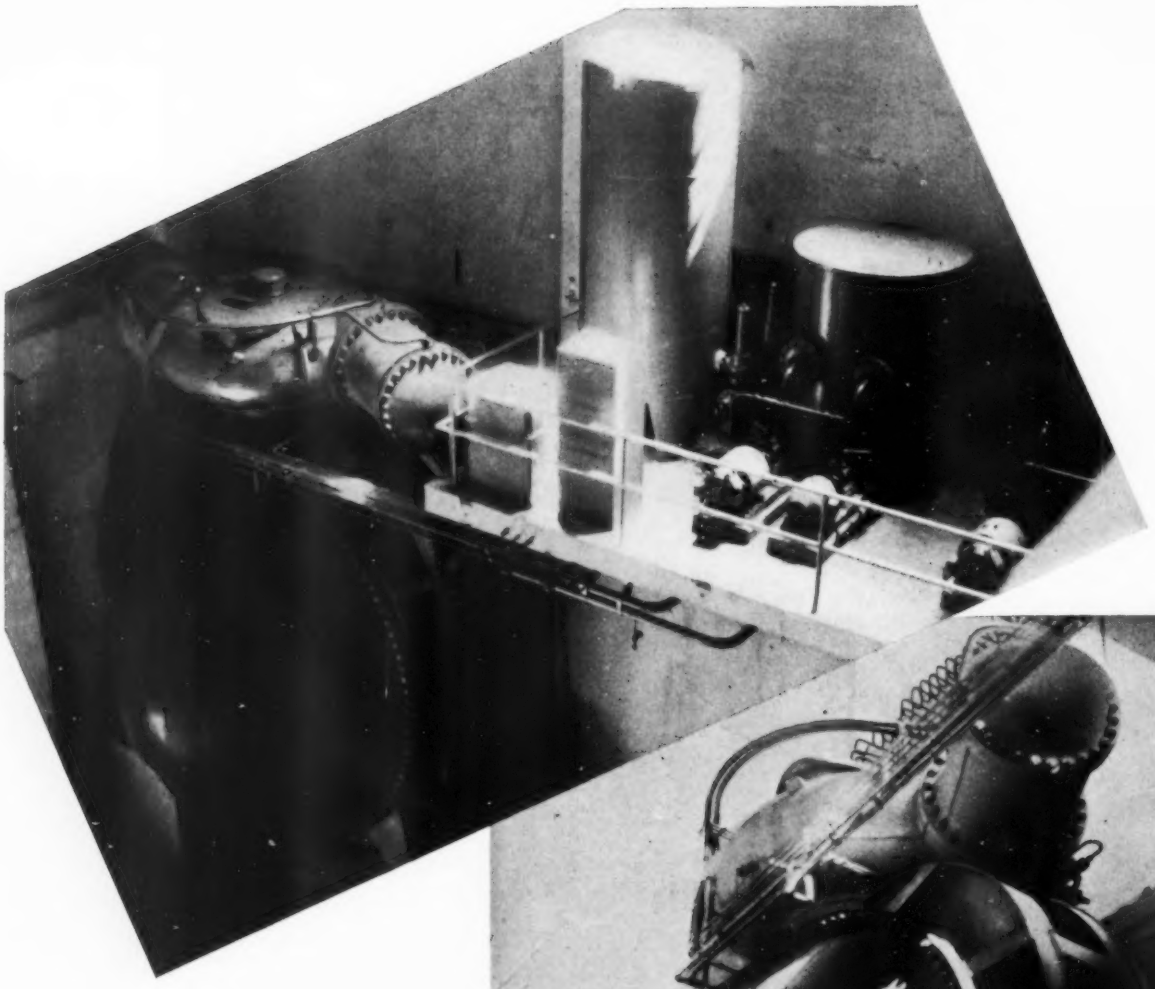
76



Butterfly valve with provisional downstream bulkhead

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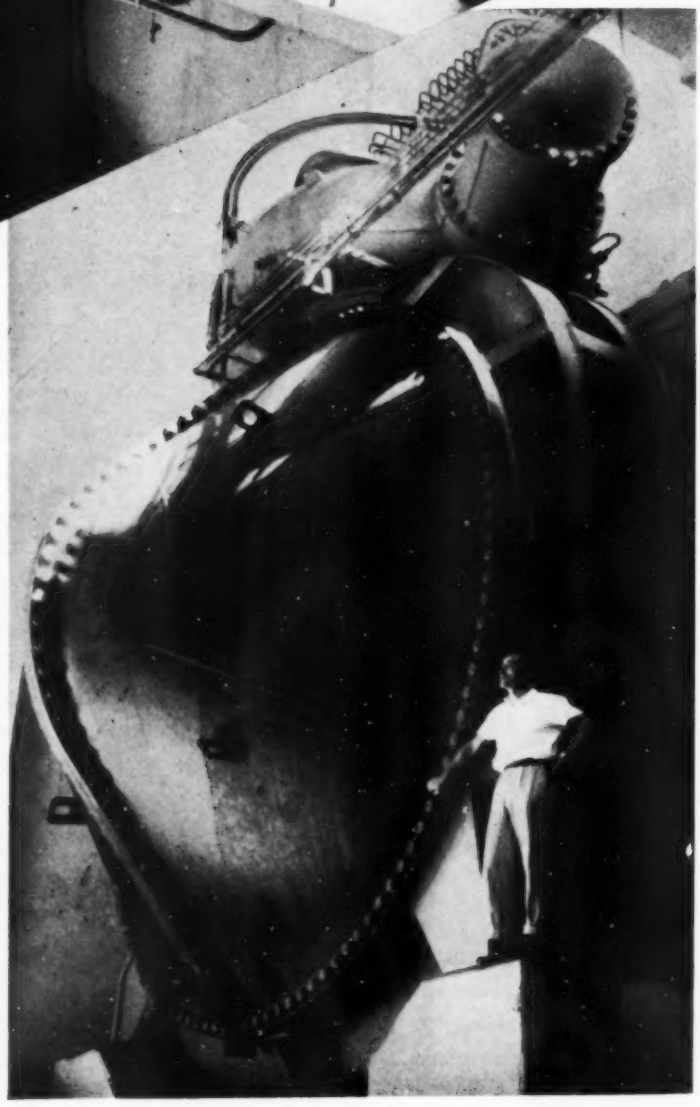

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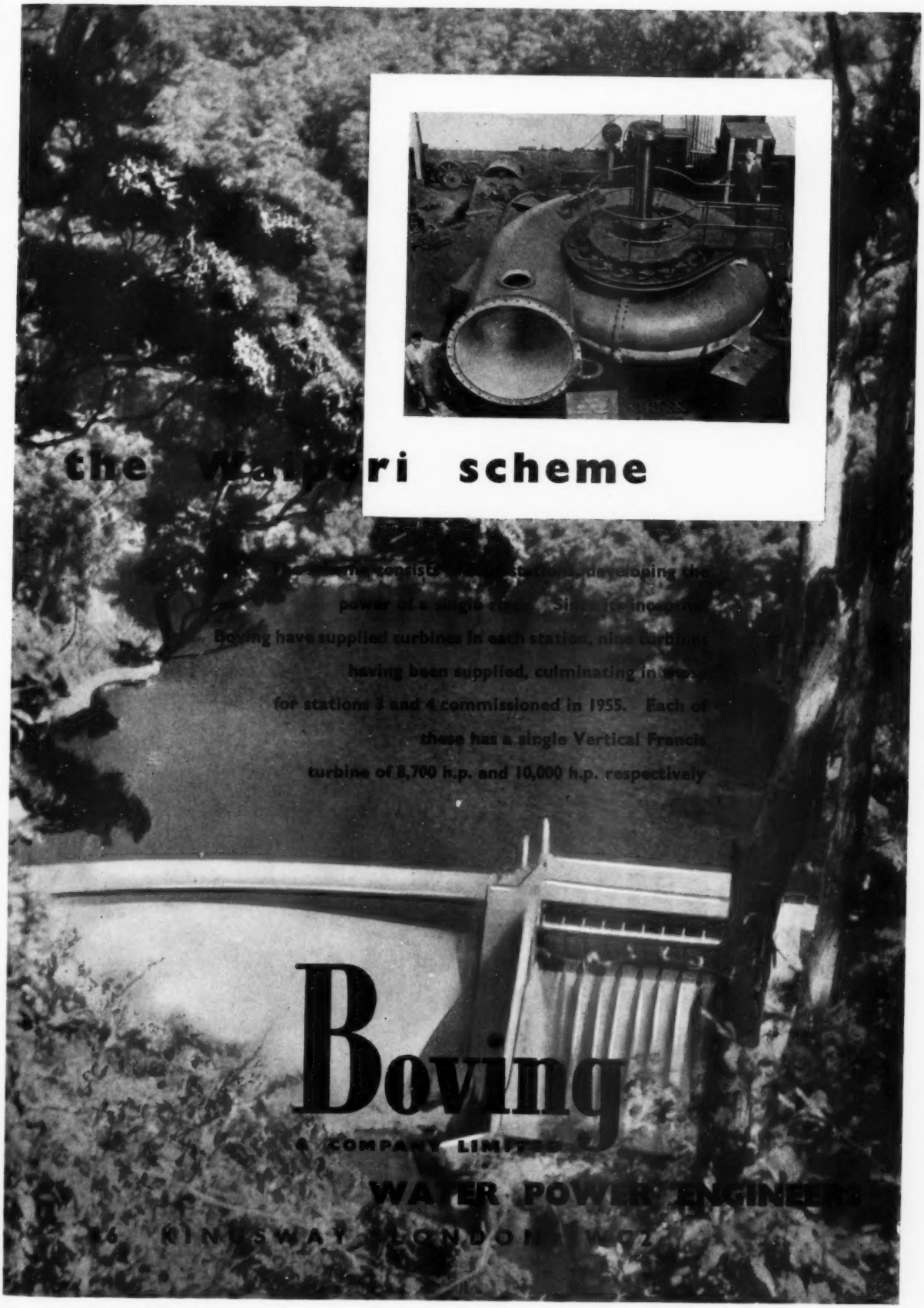
the Waipori scheme
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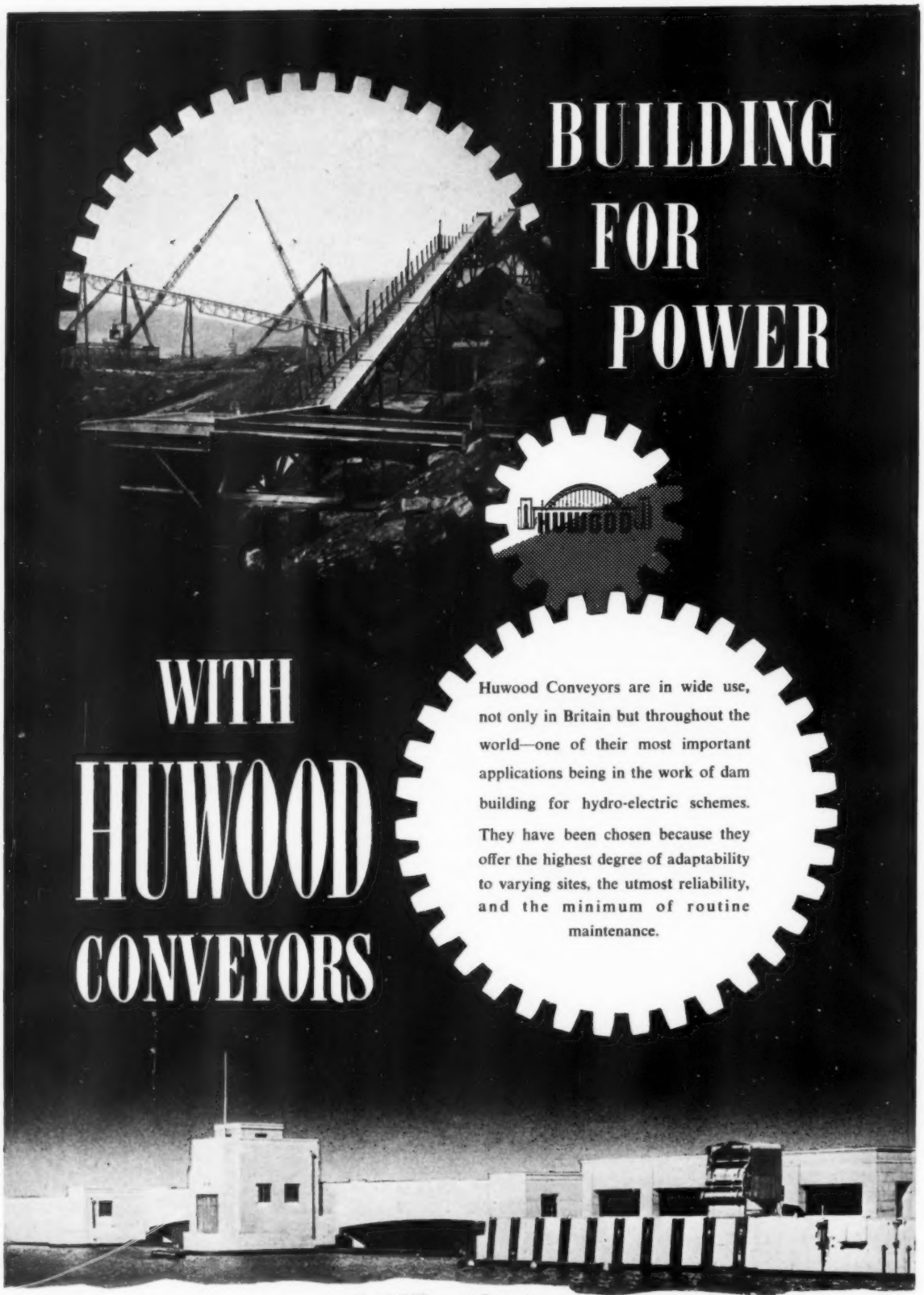
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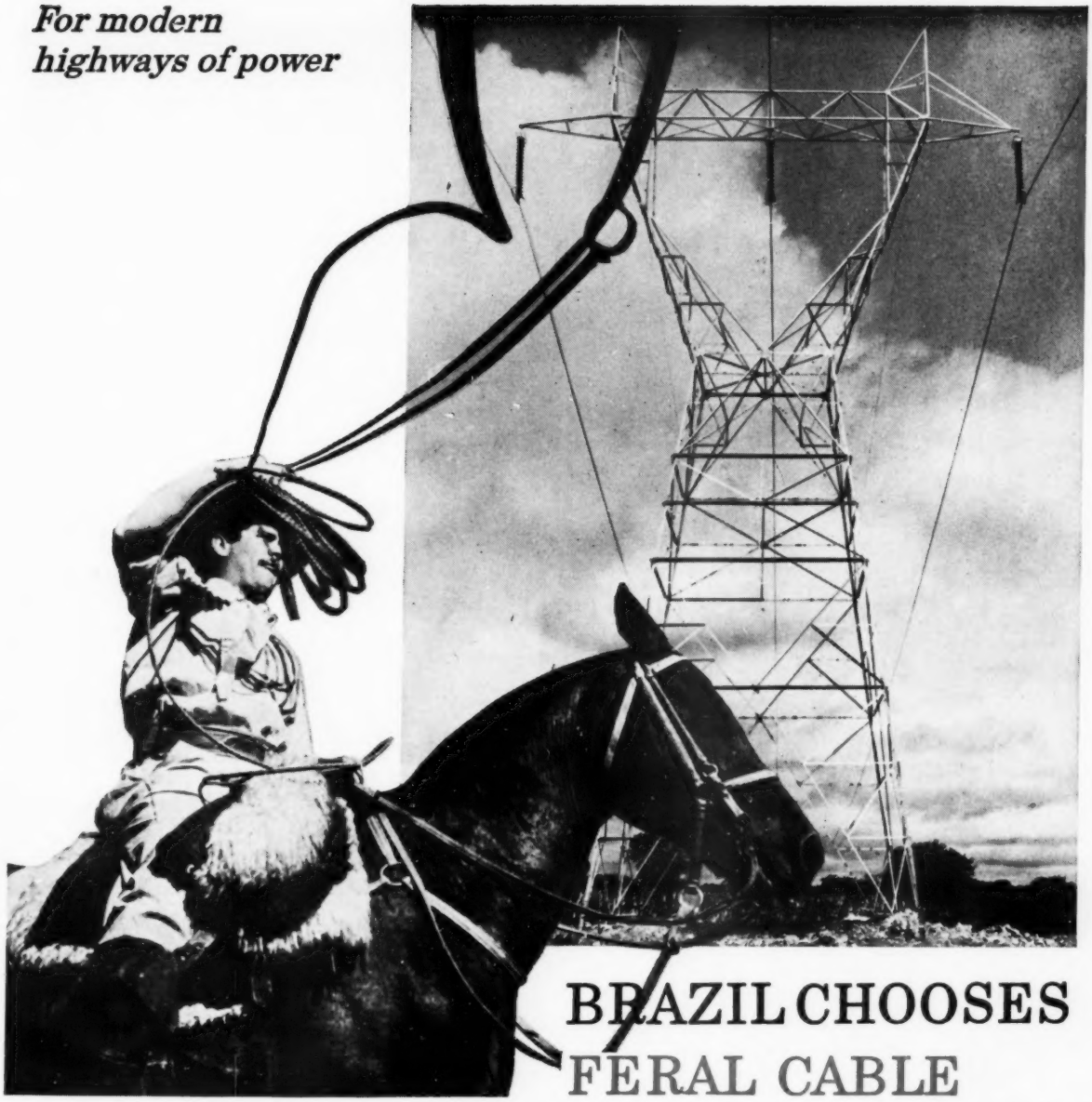
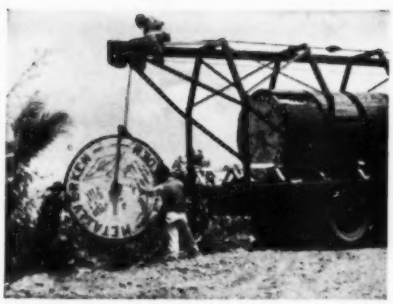


BRAZIL CHOOSES FERAL CABLE



Humid trade winds blowing in from the South Atlantic bring wealth to Brazil in the form of abundant tropical produce—coffee, cotton, cocoa, tobacco, waxes and fruits. The plentiful rainfall also means that Brazil has a big water-power potential, which is steadily being developed as a means of enabling the country to attain a more balanced economy. Among the more remarkable hydroelectric projects of the last decade has been the harnessing of the mighty Paulo Afonso falls on the Sao Francisco river. From here power is distributed over an area which includes one-fifth of the country's population. And for recent power-line extensions which practically amount to a doubling of the network Cia Hidro Eletrica do Sao Francisco again chose FERAL CABLE—*reliable* Swedish ACSR.

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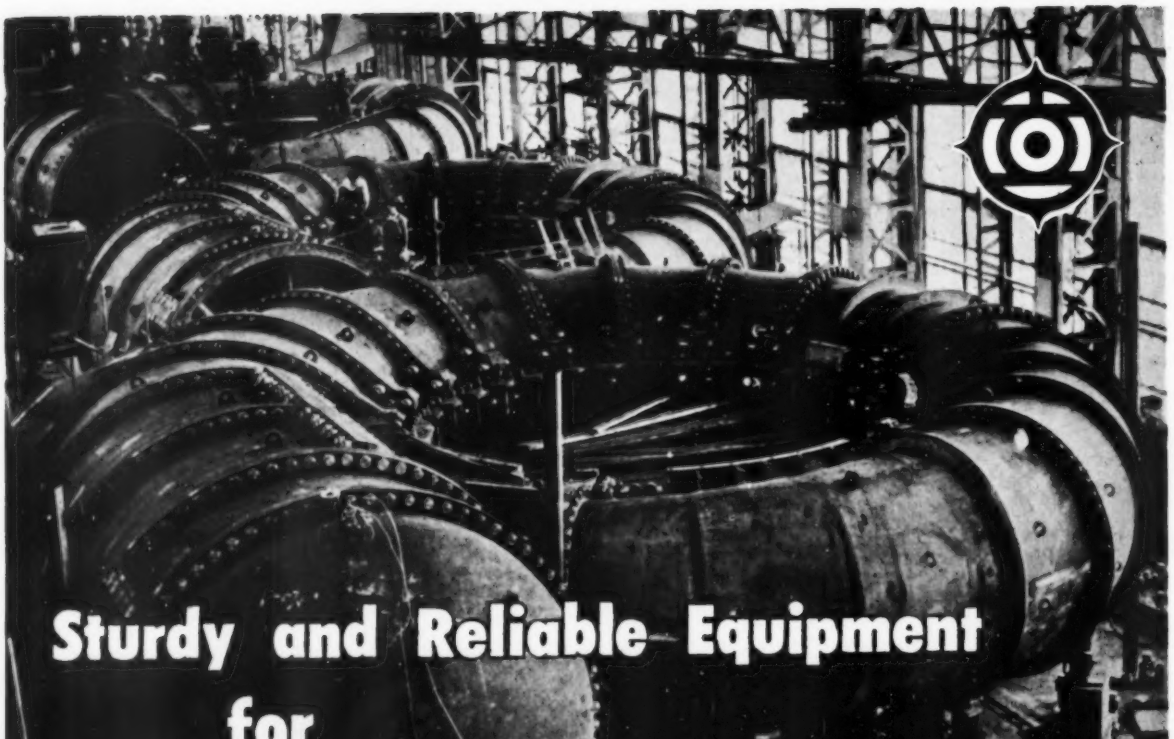

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CALAIS

BOULOGNE

The French and British power administrations, Electricité de France and the Central Electricity Generating Board, have now decided to carry into effect the plan for the exchange of power across the English Channel, which has been under discussion ever since the end of the Second World War.

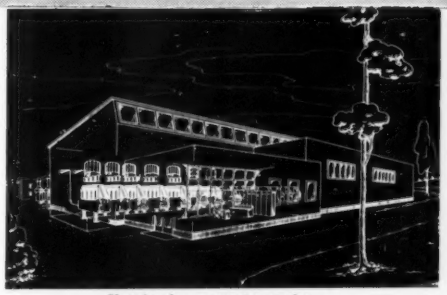
At the end of 1957 the system developed and proposed by ASEA for high-voltage direct current, which had already been adopted for the transmission of power from the Swedish mainland to the island of Gotland in 1954—the first and only project of its kind in the world—was officially accepted. The definite contract has recently been awarded to ASEA for the principal electrical equipment for the transmission in either direction of 160,000 kW, equivalent to 800 A d. c. at 200 kV. It is anticipated that the installation will be ready for commissioning by the end of 1960.

The idea behind this project is to utilise surplus power from the one country in the other country during periods of peak load. It so happens that the load peaks occur at different times of the day and during different days of the week. Despite the close proximity of the two countries, there are also certain seasonal variations in the load peaks. In addition, this project will enable both countries to postpone or even refrain from building new power stations, which would entail a considerably greater capital expenditure than the planned d. c. link.

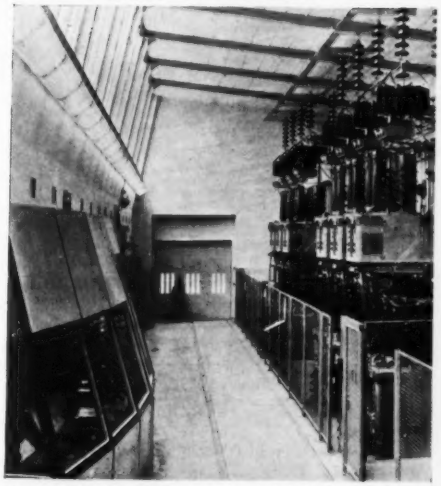
The transmission distance will be about 60 km. The converter stations are to be set up in the vicinity of Dungeness in England and Boulogne in France. It is intended to connect the d. c. link to the English 275 kV, 50 c/s grid, and to the French 225 kV, 50 c/s grid.

The ASEA d. c. system, which was selected in the face of severe competition from other alternative solutions, offers considerable advantages. The number of transmission cables can be limited, the installation costs are lower and the likelihood of mechanical faults less. Furthermore, the d. c. link will enable the network frequency of the one country to be regulated independently of that in the other country.

Västerås [redacted] Sweden



Sketch of a converter station.



Valve hall in one of the converter stations for the d.c. transmission system to Gotland, in operation since 1954.

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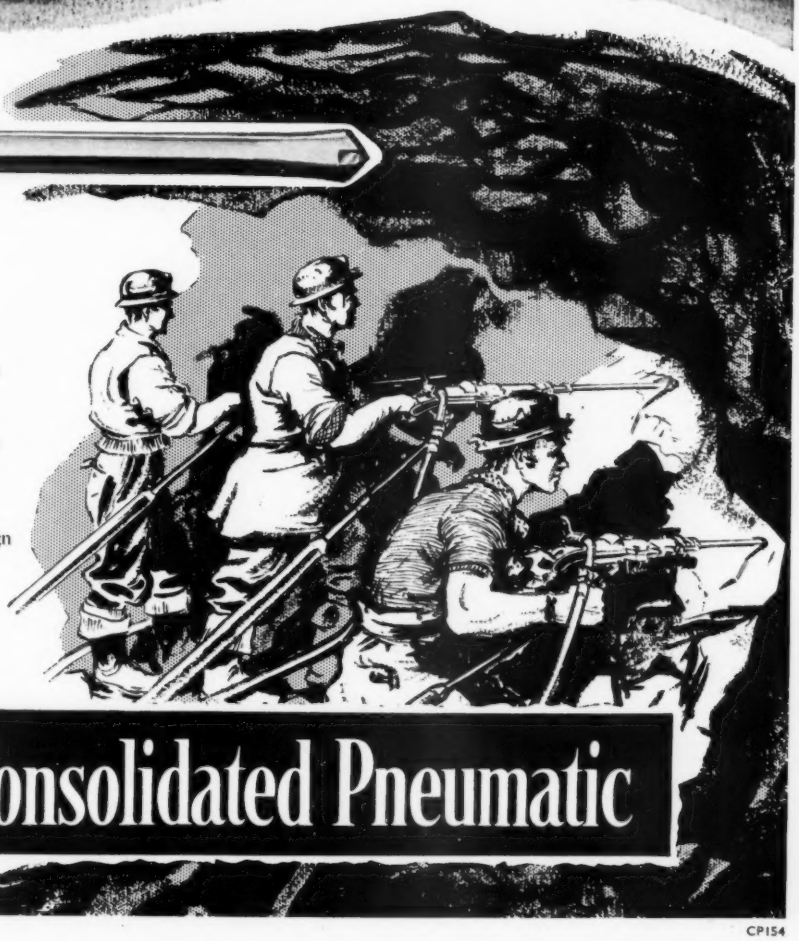

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* Easy-to-operate backhead control allows quick selection of water or air for emergency blowing.
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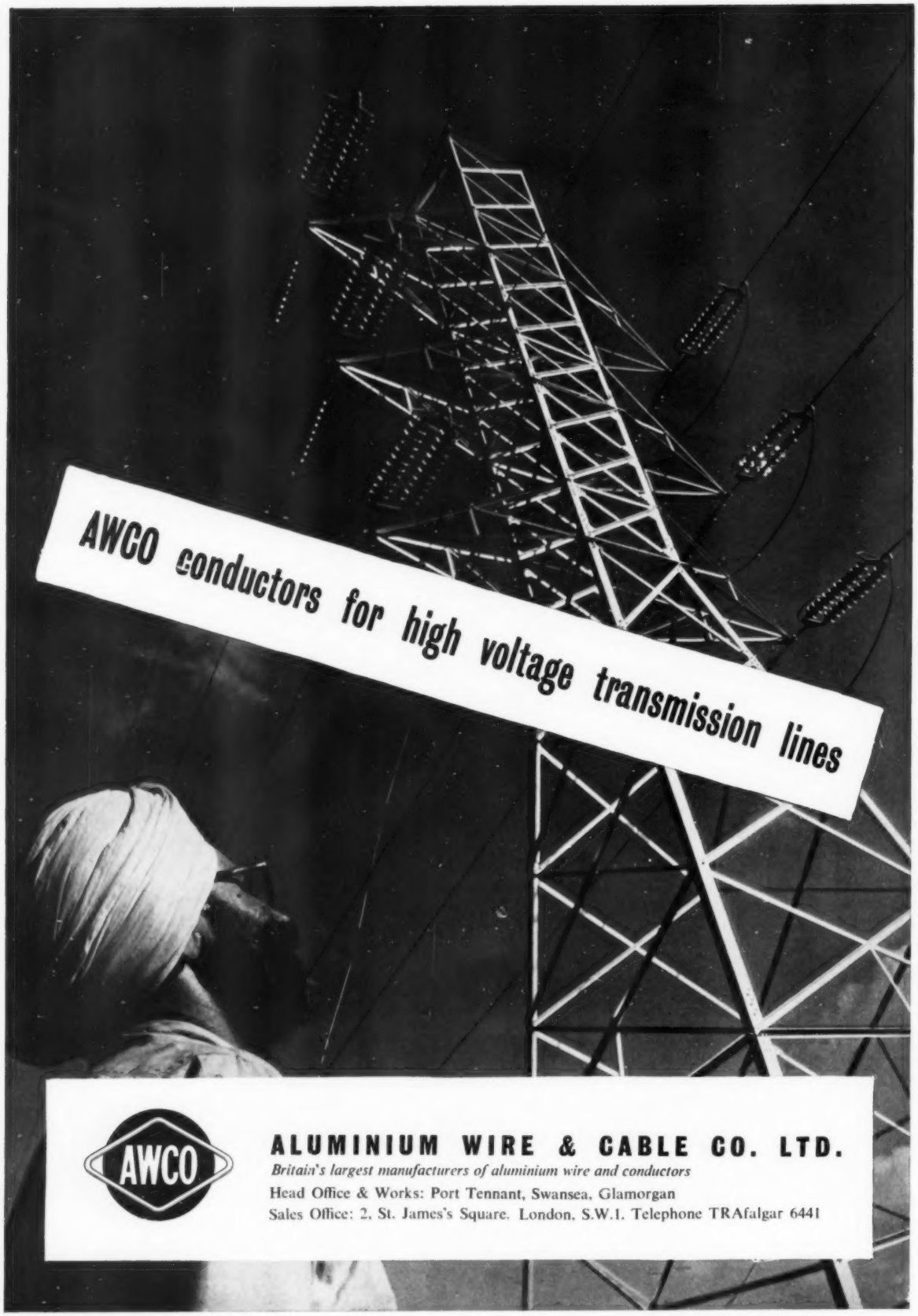
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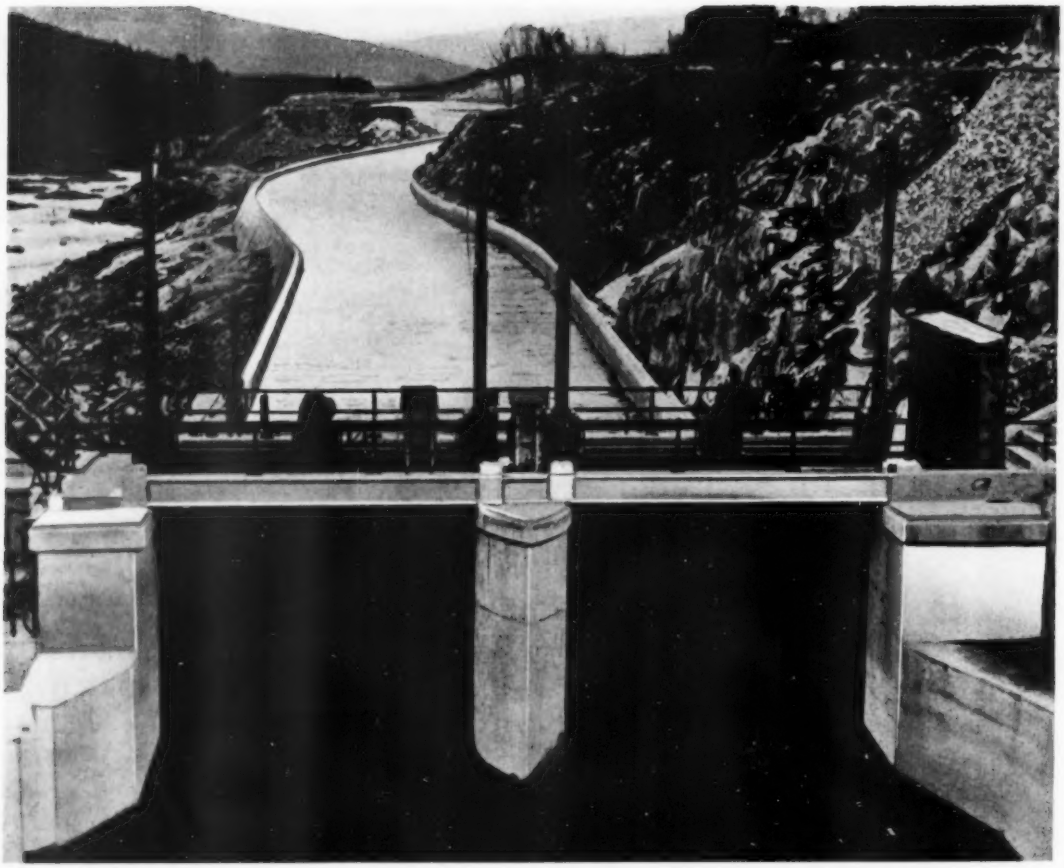

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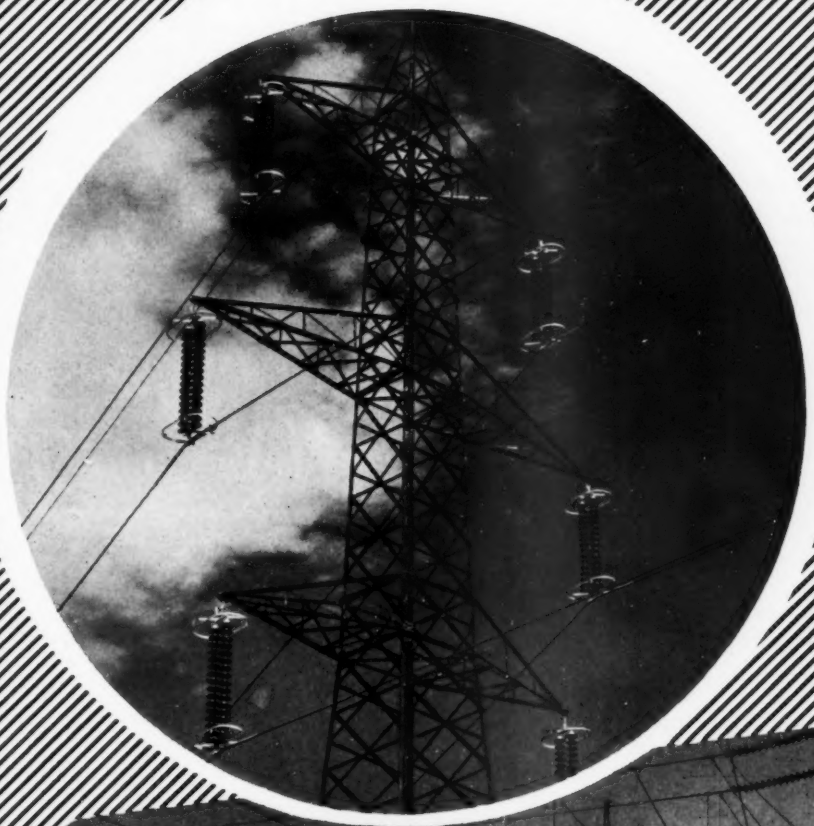

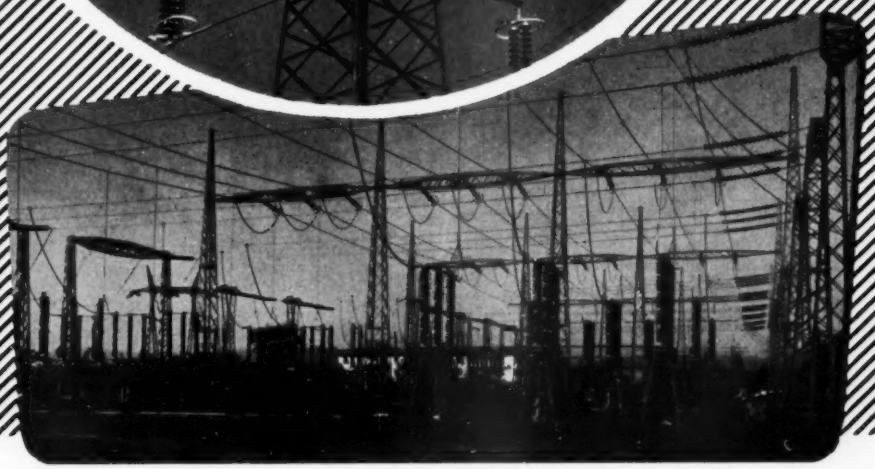

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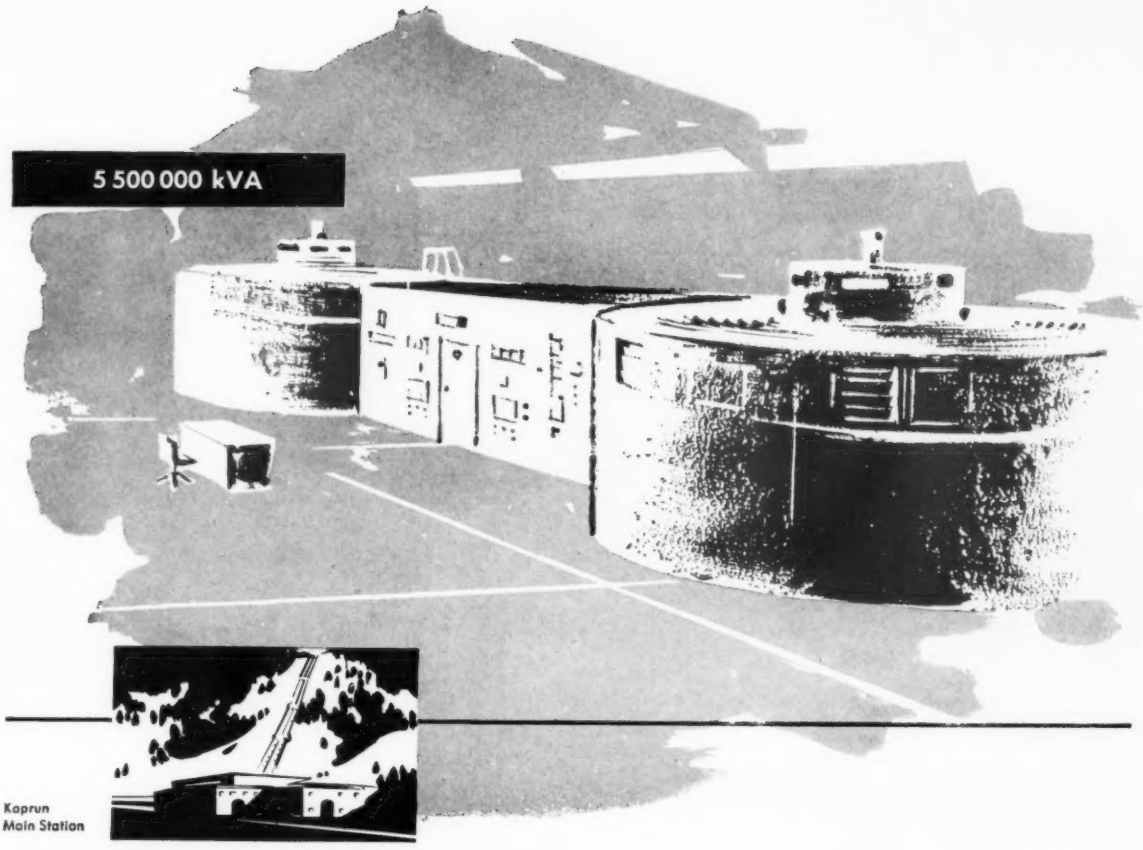
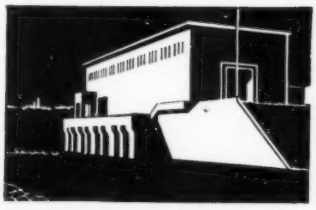
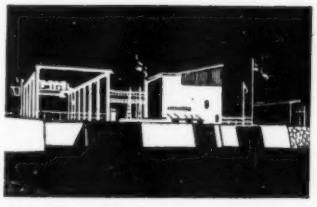


Kaprun
Main Station



Sariyar
Hydro-Electric
Power Station



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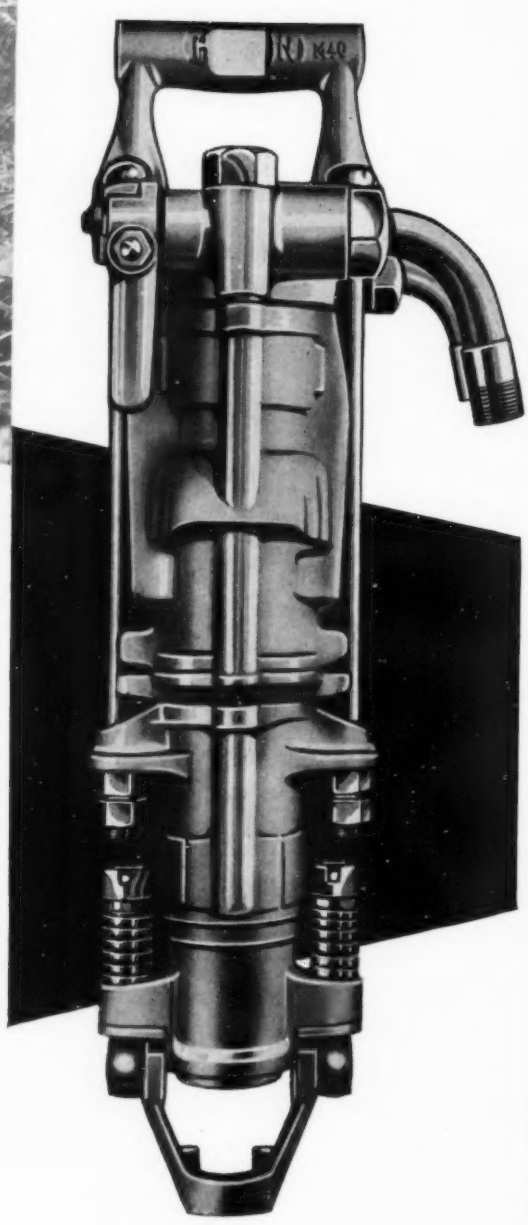

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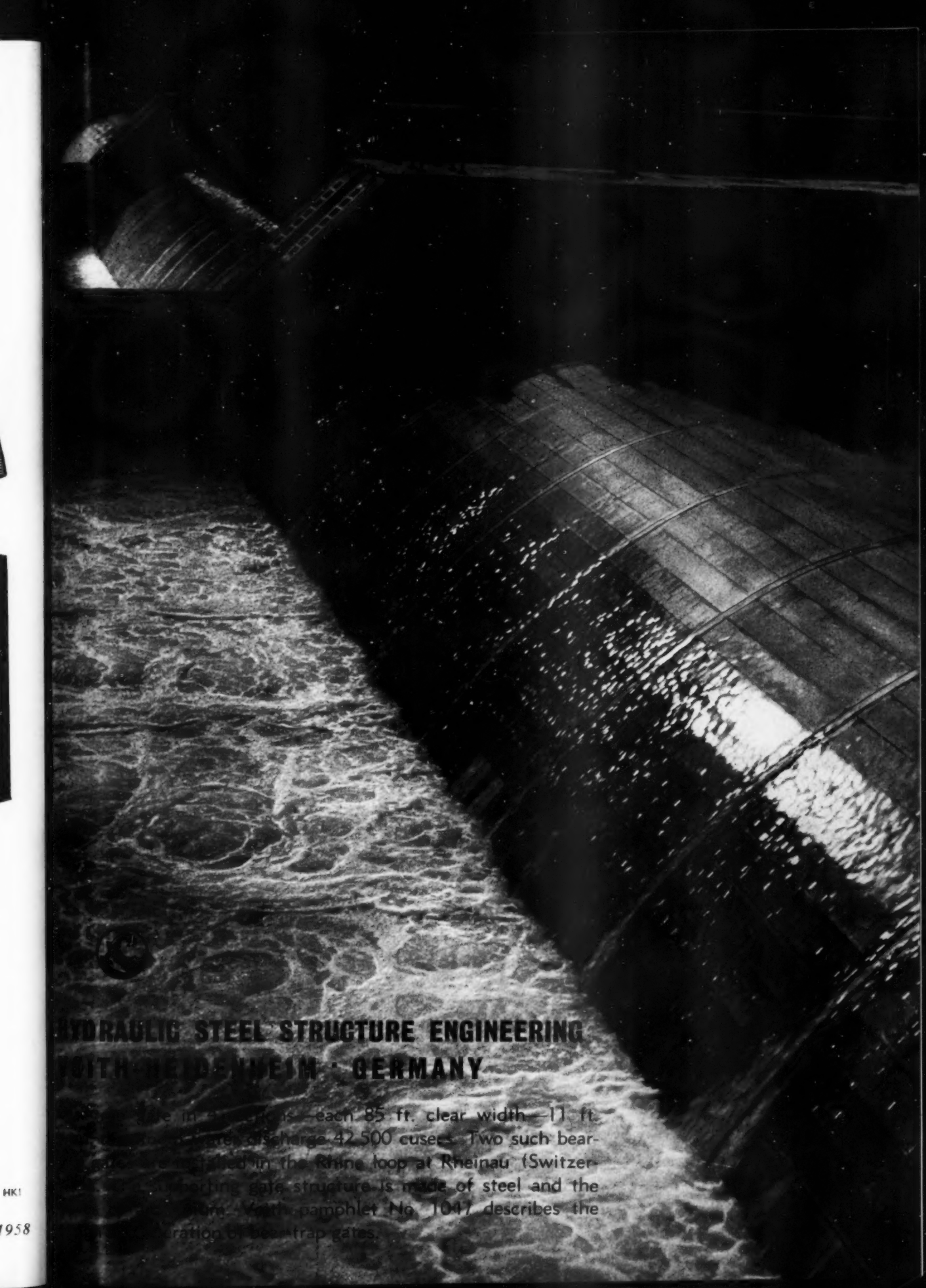

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[illegible] in [illegible] each 85 ft. clear width, 11 ft. [illegible] discharge 42,500 cusecs. Two such bear- [illegible] in the Rhine loop at Rheinau (Switzer- [illegible] gate structure is made of steel and the [illegible] Voith pamphlet No. 1041 describes the [illegible] ration of bear-trap gates.

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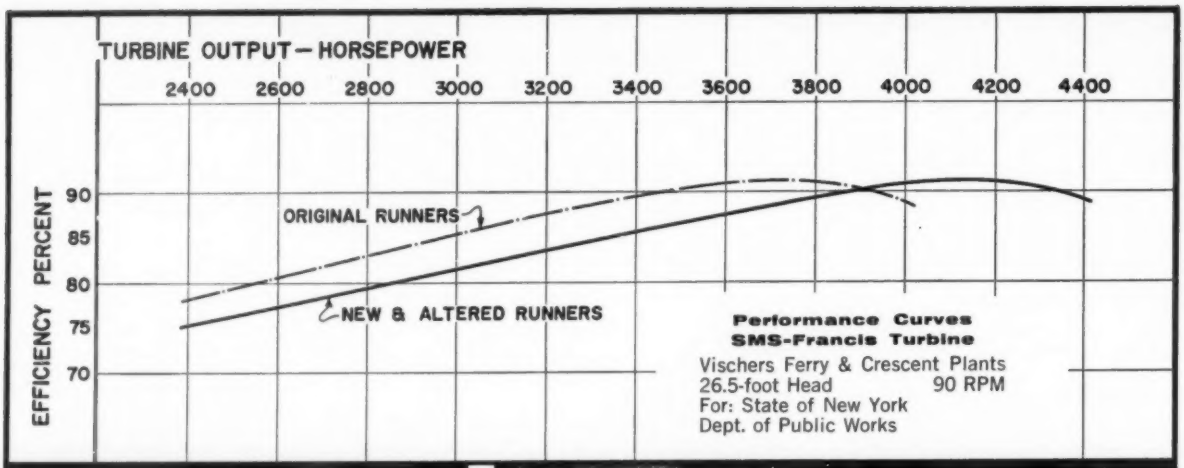
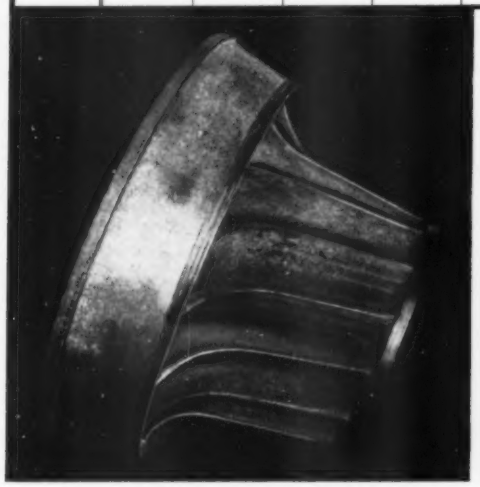
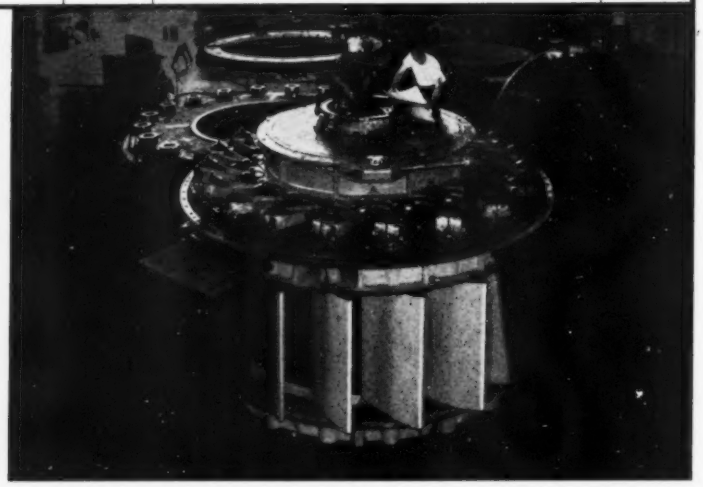




Modifed runner for Vischers Ferry unit #4. Note excellent condition of runner after long initial service.



The rebuilt distributor of Crescent unit #1 is checked during final assembly in the SMS shops.

Rehabilitation of 34-Year-Old SMS Turbines **TO INCREASE POWER 10%**

Here's how hydro plants, operating on relatively low heads, can have rehabilitated turbines to get increased power and higher efficiencies. At the Vischers Ferry and Crescent Plants on the Mohawk River north of Albany, New York, four 86-inch SMS-Francis turbines have recently been rebuilt. Their increased capacity is shown in the curve above.

These SMS runners were in good mechanical condition after an average of 34 years of service. There was no evidence of weakness or failure, no erosion, and very little cavitation. Only minor modifications were needed on the bucket and throat to reshape them in accordance with modern designs and increase their output. SMS furnished a new runner to replace the first one removed, so that by rehabilitating one unit at a time and using the runner as a replacement for the next unit, outage time was kept to a minimum and the last runner has become a spare.

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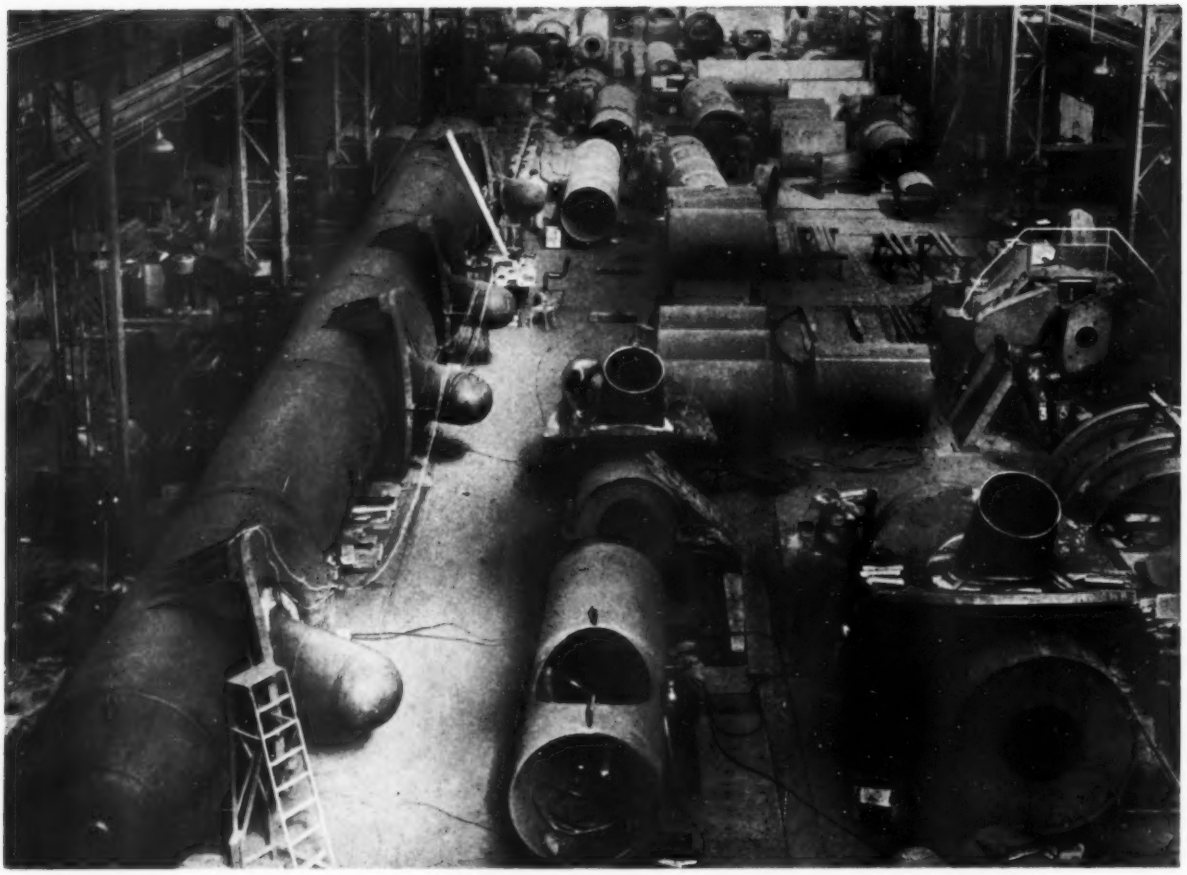
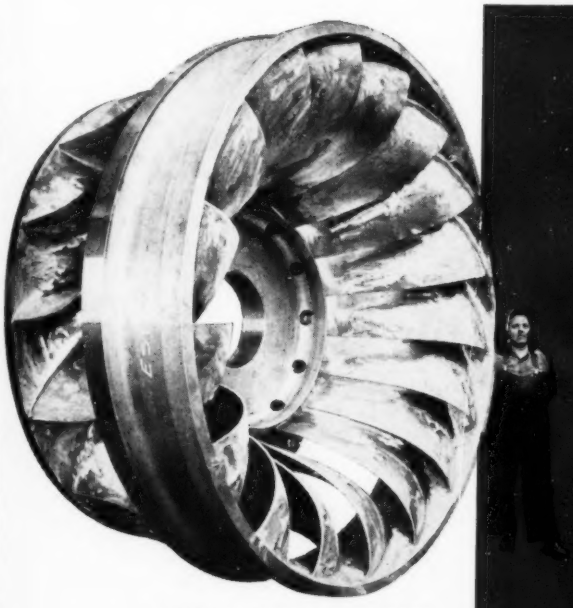
Gantry and overhead travelling cranes
Pylons, water gates



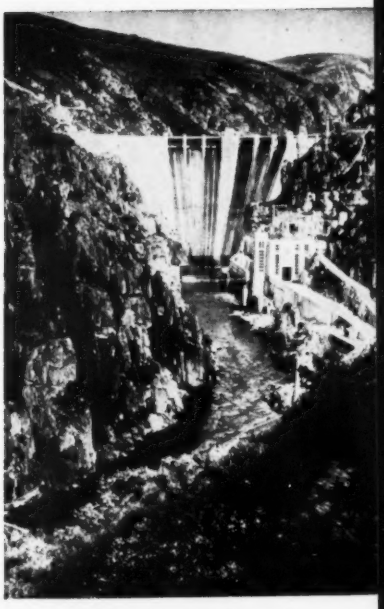
Downstream portion of the Grande Dixence Fionnay Power Station distribution pipe during tensometric shop tests. Max. wall thickness 52 mm., service pressure 96 at., test pressure 121 at.

GIOVANOLA Monthey - Switzerland

More than 20 million KMW horsepower in 50 countries



90,700 h.p. runner for one of the four KMW Francis turbines for San Esteban.



The Spanish power station of San Esteban belonging to the Saltos del Sil S.A., where the fourth KMW unit has recently been put into service.

KMW started manufacture of water turbines in 1880 and had up to the beginning of 1958 received orders for 2,765 units with a total output of 12·4 million horsepower, of which 4·7 million horsepower are Kaplan turbines. If the KMW turbines built by our licensees are included, these totals can be increased to 20 and 7 million horsepower respectively. These figures show that KMW turbines are in great demand for power stations all over the world.

Among the turbines recently delivered or ordered can be mentioned:

Number of units	Type	Client	Plant	Country	Total output h.p.	Net head feet	r.p.m.
2	Kaplan	Bålforsens Kraft AB	Bålforsen	Sweden	102,220	102	150
2	Pelton	Hjartdöla Kraftverk	Hjartdöla	Norway	142,000	1,820	428
1	Francis	A. Johnson & Co. (Canada) Ltd.	Willson	Canada	82,000	263	180
2	Kaplan	Swedish State Power Board	Järkvissle	Sweden	118,920	45	75
3	Kaplan	Bjurfors AB	Bjurfors	Sweden	101,550	64	125
3	Francis	Empresa Nacional de Electricidad S.A.	Pullinque	Chile	67,200	156	250
2	Francis	Stora Kopparbergs Bergslags AB	Trängslet	Sweden	280,000	459	214
2	Kaplan	Graningeverkens AB	Kvistforsen	Sweden	183,000	164	150
2	Francis	Stockholms Superfosfatfabriks AB	Parteboda	Sweden	42,700	105	167
2	Francis	AB Mölnbacka—Trysil—Höljes Kraftverk	Höljes	Sweden	182,800	284	187·5

AB KARLSTADS MEKANISKA VERKSTAD

KARLSTAD — SWEDEN

Representatives in Canada: A. Johnson & Co. (Canada) Ltd.,
607 Shell Tower Building, Montreal.

KMW

A. O. SMITH MULTI-LAYER PENSTOCKS

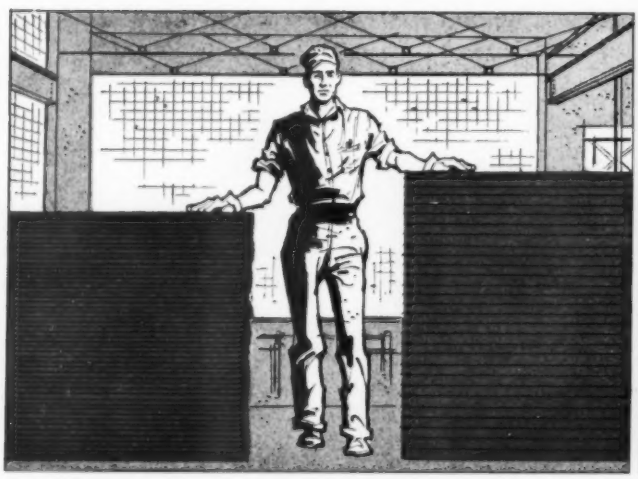
How built-up walls cut down costs

Savings start—then grow, the moment A. O. Smith MULTI-LAYER penstocks are installed. Ideally suited for high hydrostatic pressures, MULTI-LAYER's remarkable strength results from the *number* and *type* of built-up thin steel plates required to handle your job.

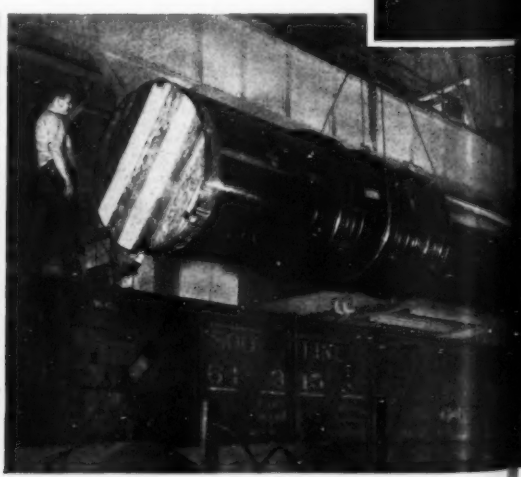
Critical material control such as this offers optimum safety and strength at lowest possible cost. Yet there's no practical limitation to the pressures that can be handled.

A complete selection of MULTI-LAYER sizes are available . . . small-diameter, thin-wall up to 30″ . . . larger diameters made to order including "wyes," "bends," and other "specials."

For aid in solving penstock problems, backed by unlimited research and production facilities, write A. O. Smith International S. A., Administrative Offices: P. O. Box 331, Milwaukee 1, Wis., U.S.A.

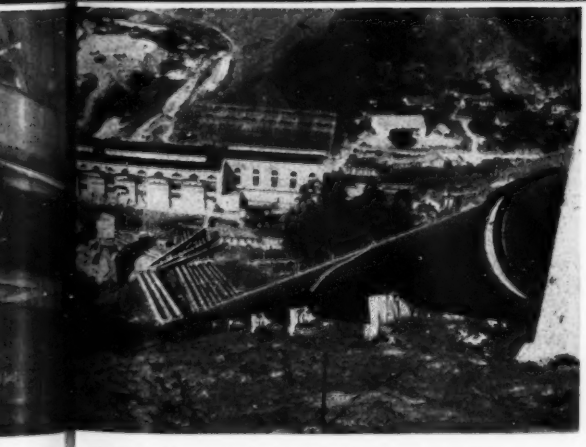


MULTI-LAYER saves up to 15% in amount of steel required for conventional *thick* plate, single wall penstocks of the same diameter. MULTI-LAYER *saves tonnage*. Layers of thin, high-tensile strength steel plate are progressively wrapped, tightened and welded around the core.



MULTI-LAYER saves in freight and cargo costs. A. O. Smith MULTI-LAYER penstocks weigh less . . . shipping costs are substantially cut. Especially important when jobs are located in remote areas. Handling and erection are faster, easier at the job site.

Manufacture of MULTI-LAYER pipe section. This modern A. O. Smith construction results in lighter, safer and more easily welded high-head penstock sections, as compared to ordinary single wall construction.



MULTI-LAYER saves by eliminating costly stress-relieving of welds in the field. Residual stresses in welds are negligible. MULTI-LAYER requires no bothersome, time-consuming handling of sections into and out of stress-relieving furnaces.

Through research A.O. Smith *... a better way*

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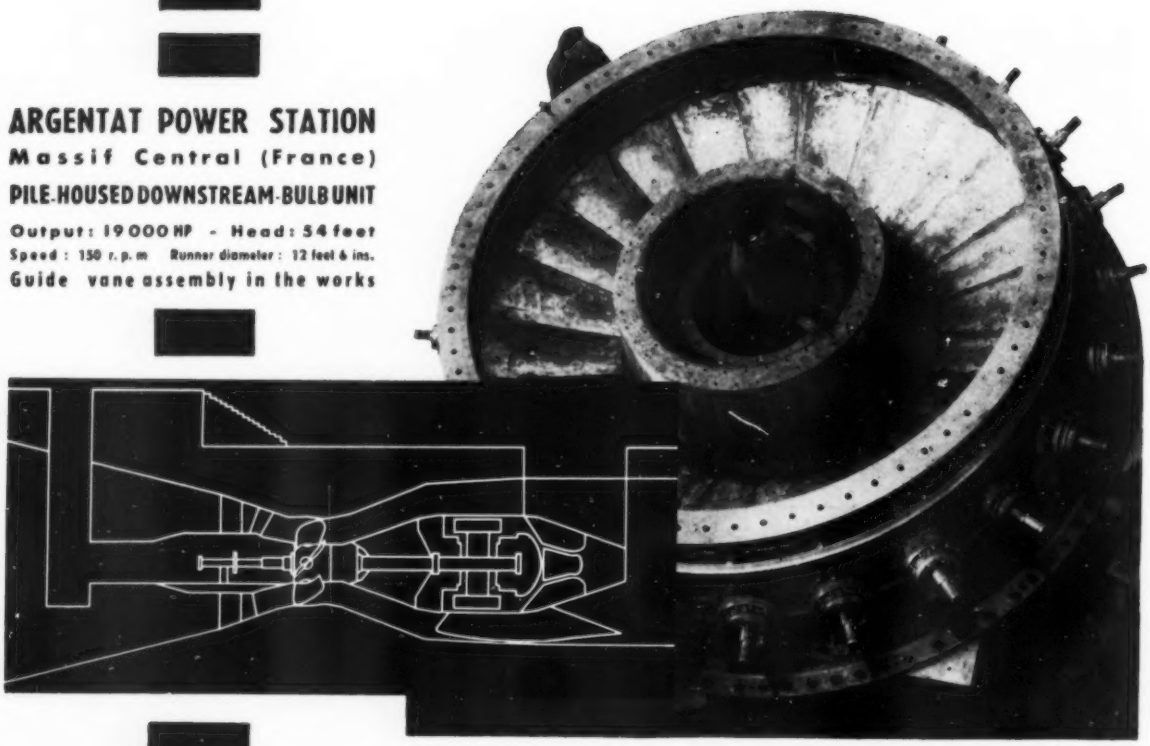
PILE-HOUSED DOWNSTREAM-BULB UNIT
Output: 19 000 HP - Head: 54 feet
Speed: 150 r. p. m Runner diameter: 12 feet 6 ins.
Guide vane assembly in the works



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F
FERRANTI

330,000 volts

transformers for Kariba

The contract awarded to Ferranti Ltd. by the Federal Power Board for the great Kariba Hydro-Electric Scheme on the Zambesi River covers two 120,000 kVA 330/234 kV 3-phase auto transformers with separate boosters; eight 60,000 kVA 330/88 kV 3-phase double-wound transformers; and two 60,000 kVA 330/33 kV 3-phase double-wound transformers. All the transformers will be provided with on-load tap changing gear. The placing of this large and important overseas transformer contract emphasises the world-wide confidence in Ferranti Ltd. as builders of fine transformers.

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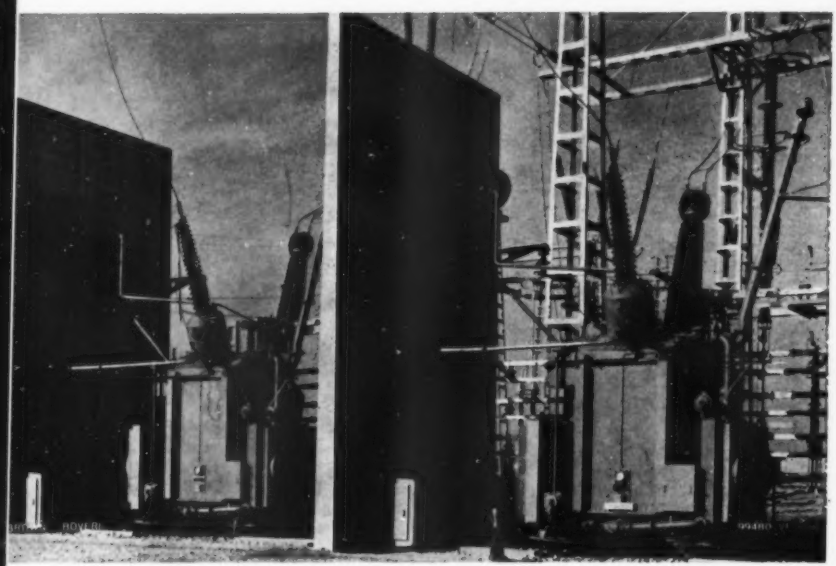
London Office: KERN HOUSE · 36 KINGSWAY · W.C.2

FERRANTI

FT 208/2

BROWN BOVERI

400-kV Equipment



Electricité de France, Creney Substation
"Alpes-Paris" 380-kV transmission system. The illustration shows two of the three single-phase auto-transformers each rated 300/3 MVA; 380 ± 5% / 225 / 10.5 kV; designed for transport by rail and impulse tested at 1450 kV.

Transformers

Building and in service:

Krångede A.B. Sweden
Two banks of transformers each rated 130 MVA; 13.85/400 kV.

Electricité de France
One bank of transformers rated 300 MVA; 380 ± 5%/225/10.5 kV.

Rheinisch-Westfälisches Elektrizitätswerk (RWE) Germany
Two single-phase transformers each rated 660/3 MVA; $400 / \sqrt{3} / 231 / \sqrt{3}$ ± 18%/30 kV.

Kraftwerke Hinterrhein AG. Switzerland
Two banks of transformers each rated 400/280/2 × 73.5 MVA; 420/201 ± 28 × 3.75/2 × 10.5 kV.

Imatran Voima Oy, Finland
Four single-phase transformers each rated 420/3 MVA; 380/238/20 kV.

Switchgear

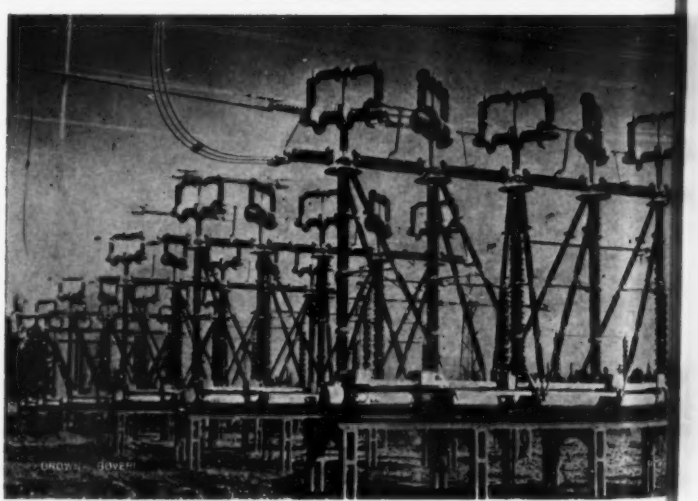
Building and in service:

Kungl. Vattenfallsstyrelsen, Sweden
10 three-pole air-blast circuit-breakers
33 three-pole isolating switches
33 current transformers

Imatran Voima Oy, Finland
12 three-pole air-blast circuit-breakers

Rheinisch-Westfälisches Elektrizitätswerk (RWE) Germany
2 three-pole air-blast circuit-breakers

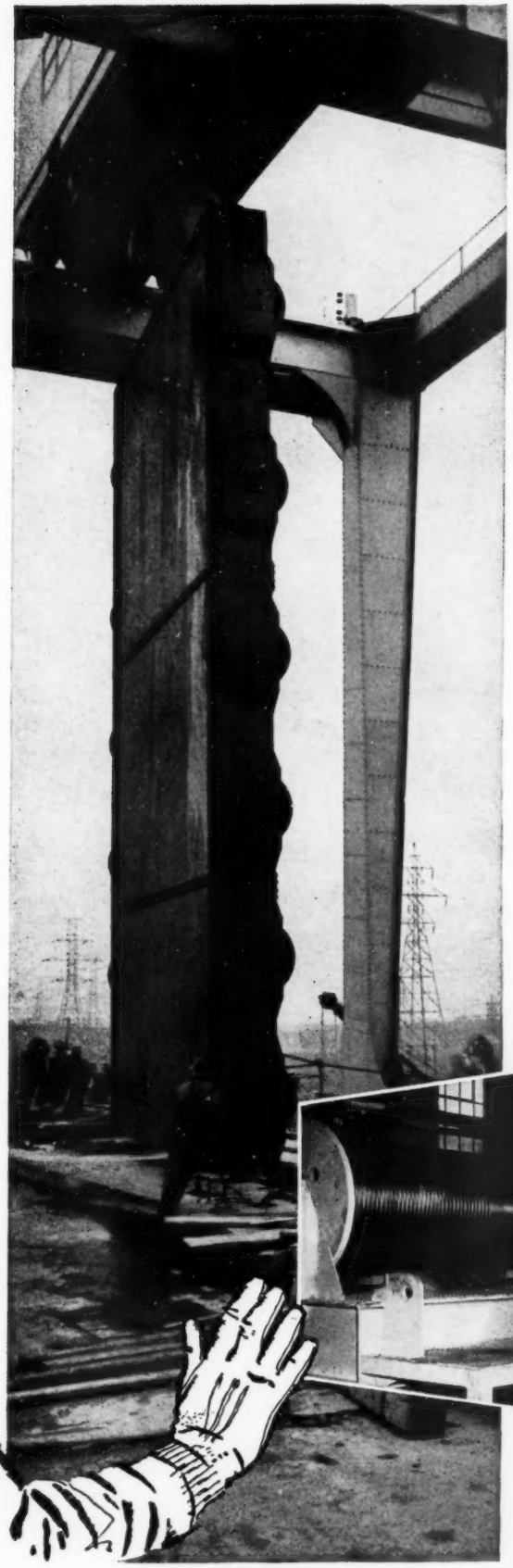
Kraftwerke Hinterrhein AG. Switzerland
2 three-pole air-blast circuit-breakers



Kemijoki Oy, Helsinki, Finland
Petäjäskoski switching station, 380-kV-air-blast circuit-breakers type DCVF 380 m 10 r

BROWN, BOVERI & CO., LTD., BADEN (SWITZERLAND)

Representatives in most countries



Intake Gates

Illustrated is a completely assembled Intake Gate being lowered into position. This is one of twelve built by Canadian Vickers for the Pumped Storage Scheme part of the Sir Adam Beck No. 2 Development of the Hydro Electric Power Commission of Ontario, at Queenstown.

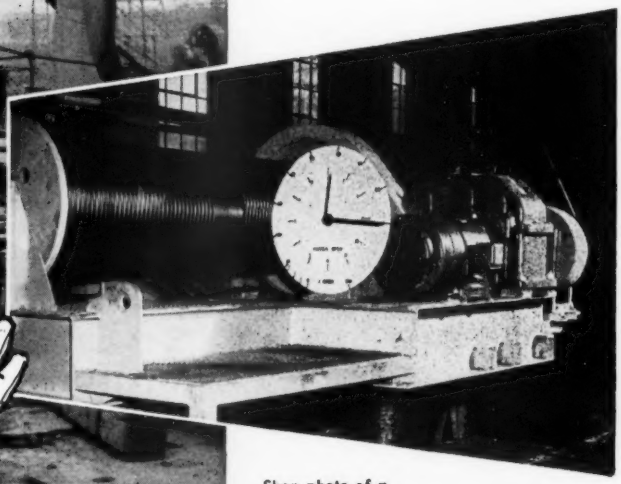
Shop welded in sections and riveted together in the field, these Gates are designed for a clear opening of 16′-8¾″ wide and 34′-9″ high and subjected to a maximum head on the sill of 77′-6″.

Canadian Vickers designs and builds all types of Water-Way and Water-Power equipment. If it is Vickers-built . . . it's right.

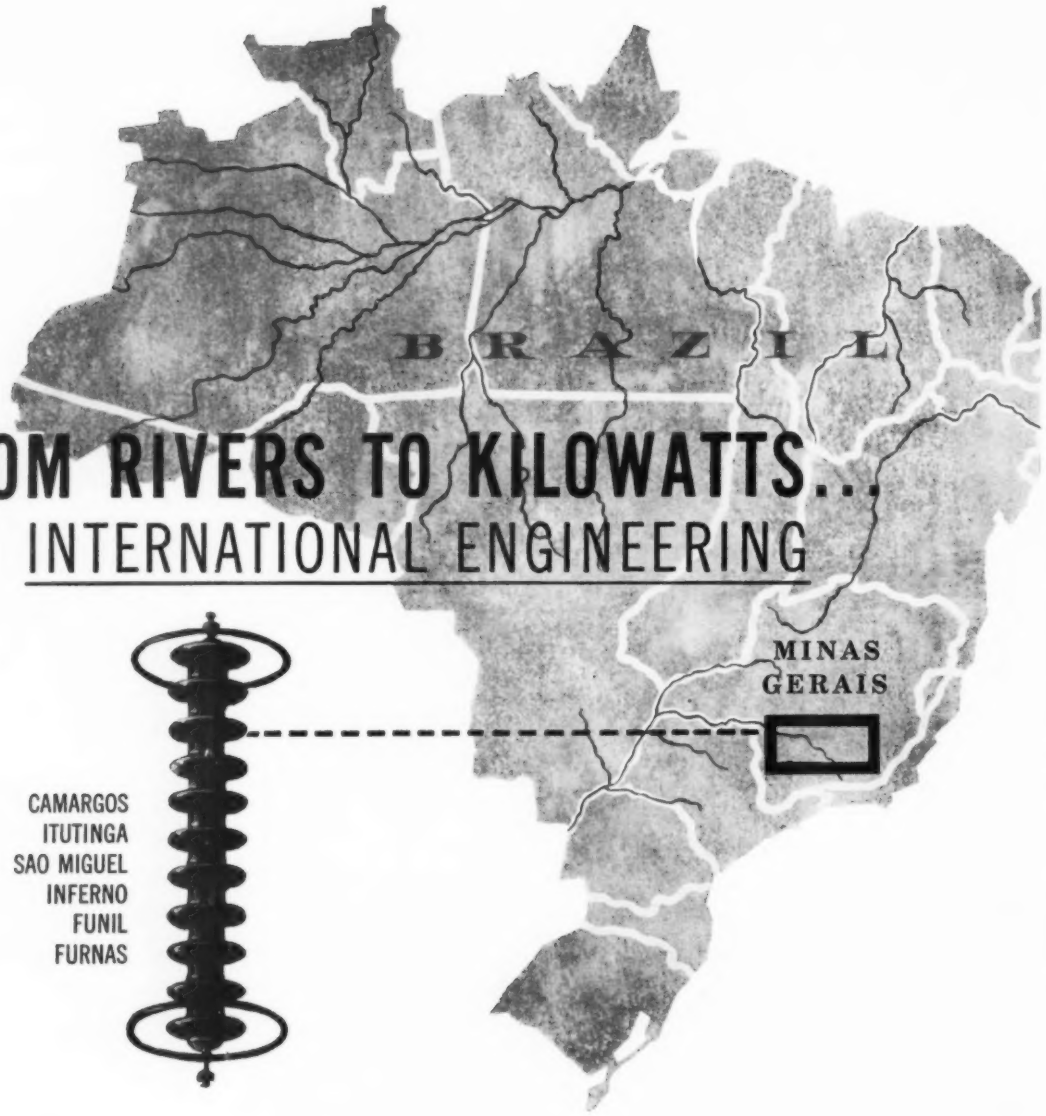
CANADIAN VICKERS LIMITED

MONTREAL TORONTO

CANADIAN MEMBER OF THE VICKERS GROUP



Shop photo of a Vickers-built Hoist.



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VIA INTERNATIONAL ENGINEERING

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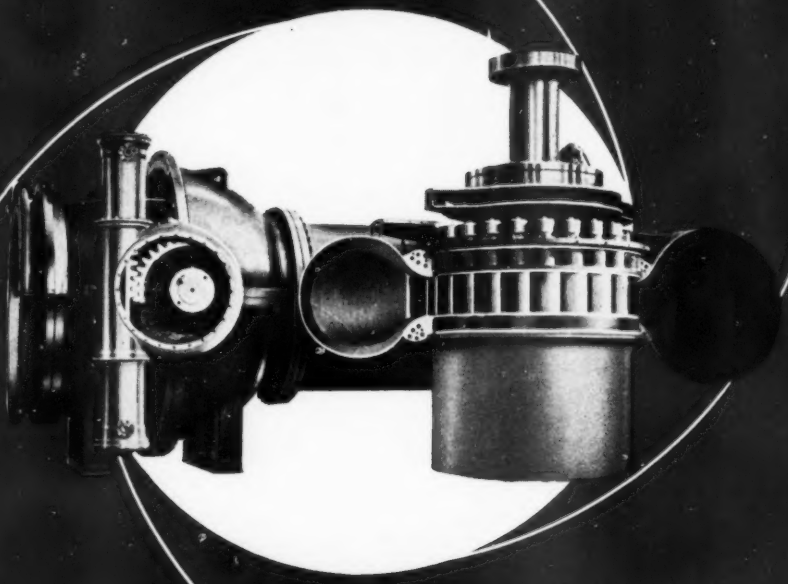
FOR WATER POWER PLANTS



Turbines of the Francis, Kaplan and Pelton type for various heads and outputs, transformers, generators, outdoor and indoor switch-gear, control-rooms, etc.

The world-known ŠKODA and ČKD trade-marks guarantee the first-class technical design and quality of equipment supplied by us. We give integral guarantees for complete deliveries.

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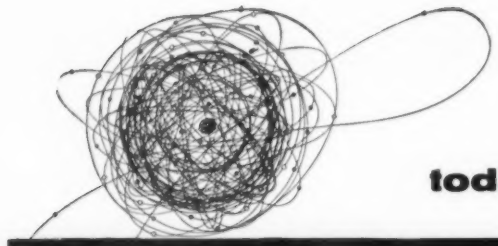
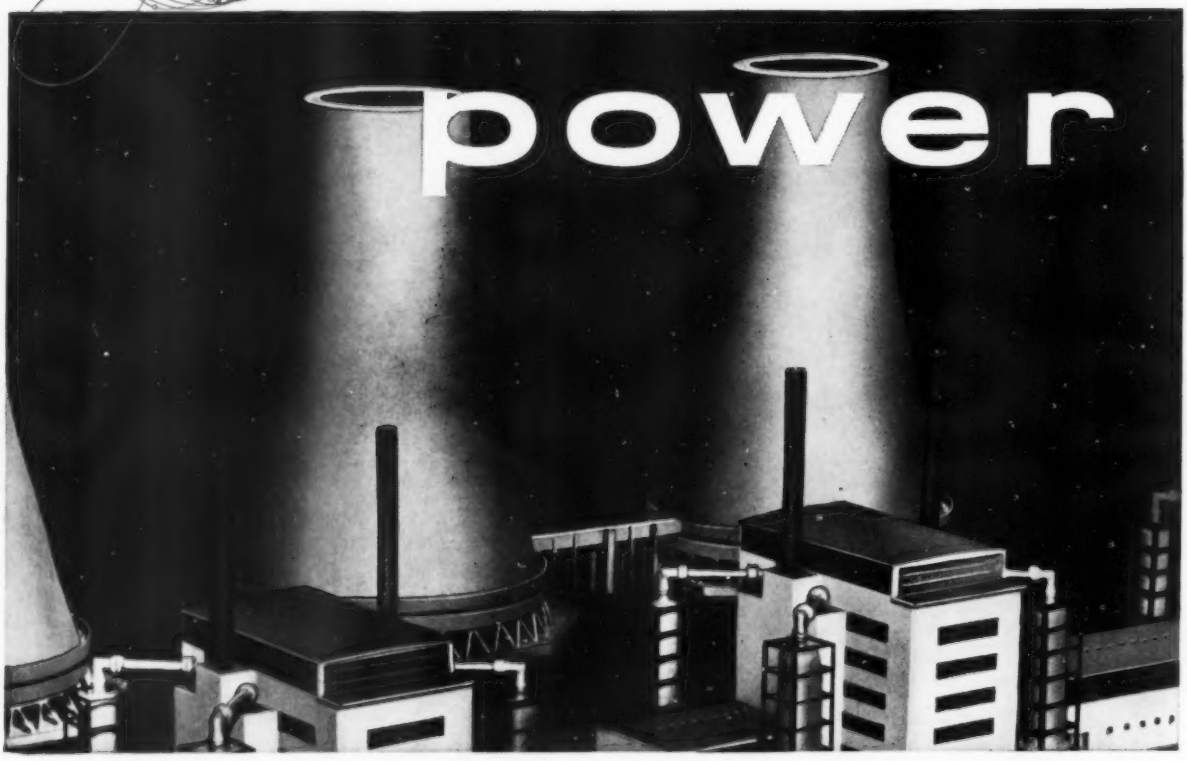


today's cables for tomorrow's



HENLEY'S CONTRIBUTION TO THE NEW ATOMIC POWER STATION AT CHAPELCROSS

Nothing is more vital to Britain's future economy than the development of the peaceful atom . . . greater source of controllable power than man has ever before known. Henley's are proud to be associated with the phase of this progress now being planned for Chapelcross, Dumfriesshire. Their contribution will be cables, the arteries along which will course the power that is Britain's life blood . . . and the small but vital cabling for the plant's control and auxiliary circuits. Over 125 miles* in all, representing a £½ million contract.

The shape of Britain's future. How Chapelcross Atomic Power Station will look on completion. We are indebted to Mitchell Construction Company, Peterborough, for permission to reproduce this photograph of part of the model of the project.

* *65,000 yards of* H.T. *and* L.T. *Paper Insulated* F.I.N.D. *type cables; 131,500 yards of* V.R. *cables; 27,000 yards of Mineral Insulated Cables. The contract is scheduled for completion by September, 1959.*

CONSULTANTS:
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32 VICTORIA STREET, LONDON, S.W.1

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VeVeY

Complete

equipment for Hydro-electric Power Plants

Turbines

governors

penstocks

valves

travelling cranes

Ateliers de Constructions Mécaniques de Vevey S.A.

1471 E

This machinist is "miking" a disc for one of the largest butterfly valves ever built – 192" diameter. Newport News built 3 such valves, each weighing 446,000 lbs., for the Ross Power Plant, Skagit Project, Department of Light, City of Seattle, Washington. Designed for a water flow of 3,620 cu. ft. per sec., and a hydrostatic pressure of 290 psi., these valves were shop tested by Newport News at 450 psi. They are hydraulically operated with oil at 1,500 psi. pressure.

Birth of a **200-ton** Butterfly

This disc for a 16-foot butterfly valve reflects two basic advantages of Newport News fabrication...

First, it exemplifies the *careful attention* Newport News craftsmen give to every detail. And secondly, it attests to the *quality* with which Newport News produces in massive equipment for public utilities and allied industries... due to Newport News' high integration of skill and production facilities.

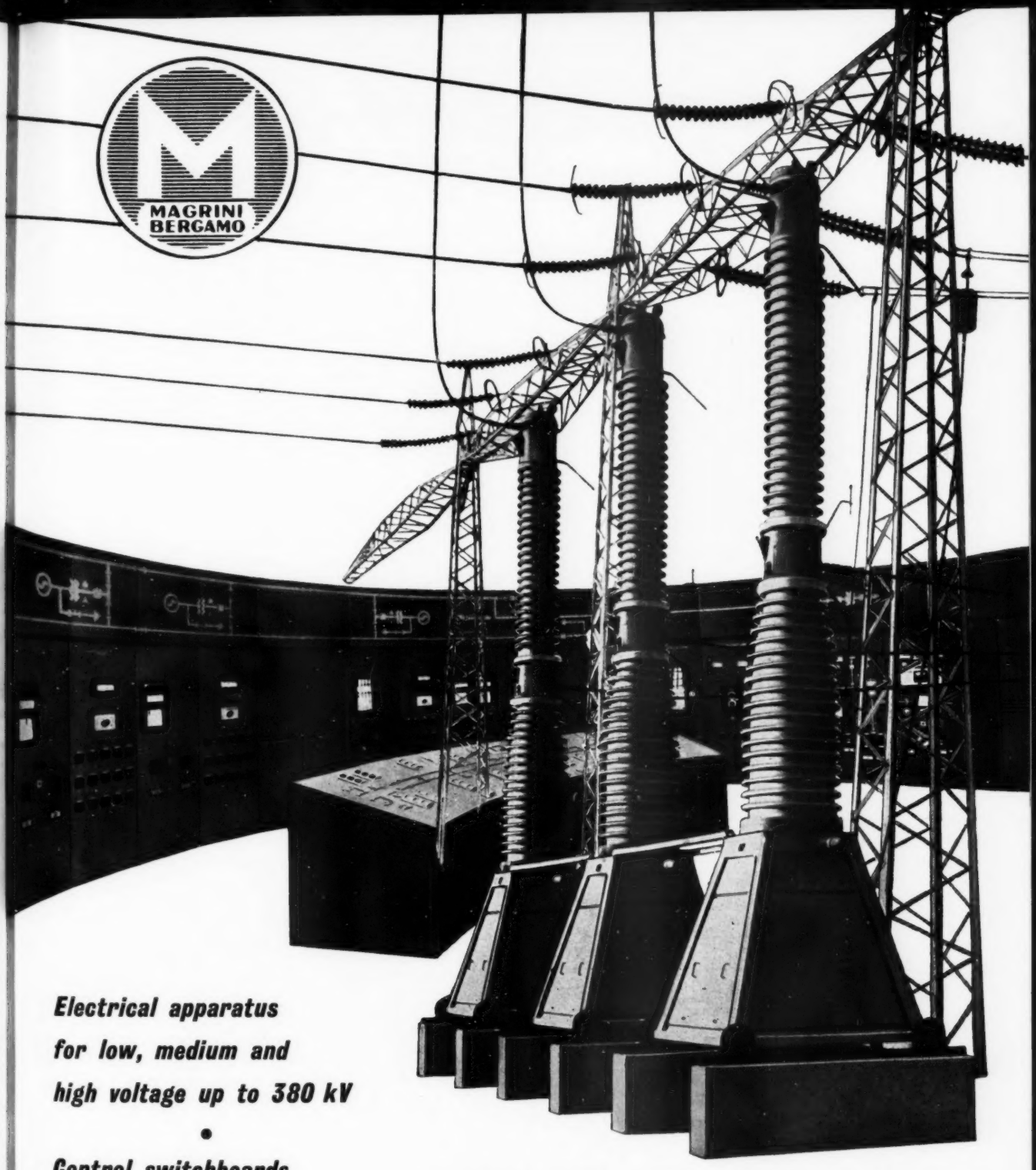
Additional advantages accrue to customers from extensive work conducted in Newport News' testing laboratories on problems related to water power equipment.

Avail yourself of the engineering talent, along with the specialized production techniques and the skill of Newport News craftsmen operating vast steel fabricating shops, five huge machine shops, drop forging and die facilities along with acres of brass, iron and steel foundries.

Let us bid on equipment for your present or future projects. If you are not familiar with the way Newport News can help you, write for our illustrated booklet entitled "Water Power Equipment" ...it's yours for the asking.

NEWPORT NEWS

Shipbuilding and Dry Dock Company
Newport News, Virginia



Electrical apparatus
for low, medium and
high voltage up to 380 kV

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and control desks

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metal clad switchgear

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Lightning arresters

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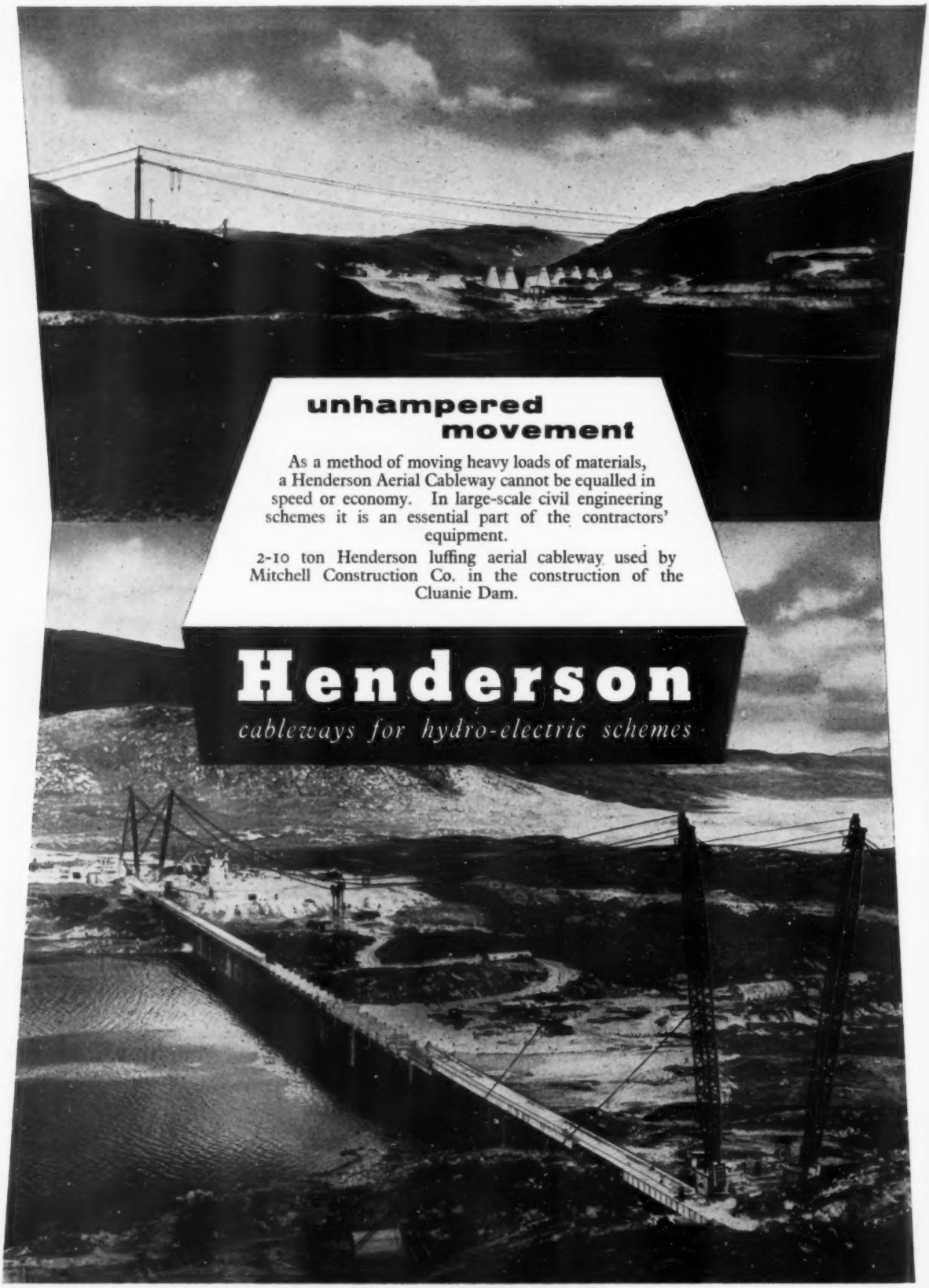
unhampered
movement
As a method of moving heavy loads of materials, a Henderson Aerial Cableway cannot be equalled in speed or economy. In large-scale civil engineering schemes it is an essential part of the contractors' equipment.
2-10 ton Henderson luffing aerial cableway used by Mitchell Construction Co. in the construction of the Cluanie Dam.
Henderson
cableways for hydro-electric schemes

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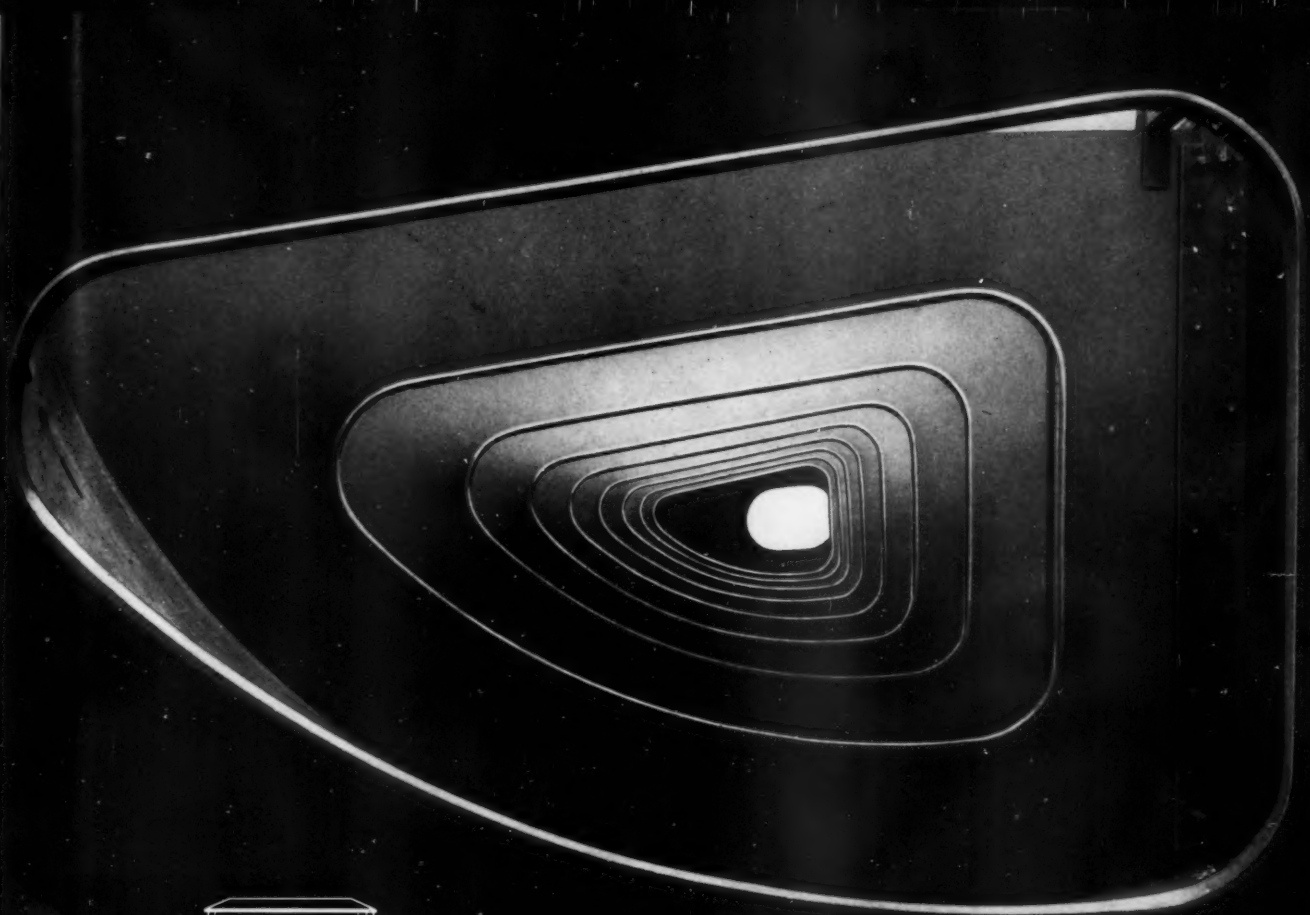

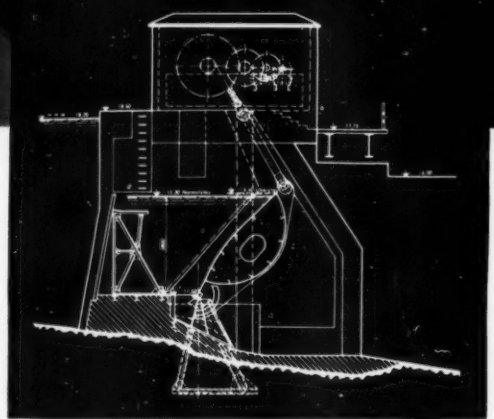

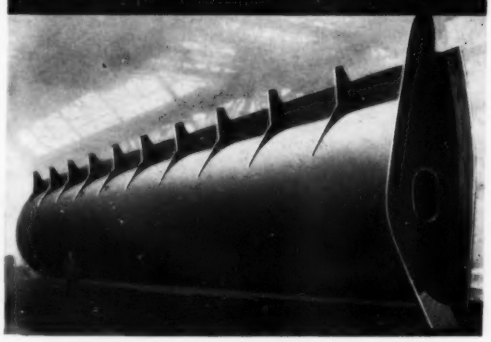

Above: Fishbelly flap, inside view
Centre: Section of a weir with fishbelly flap
Below: Fishbelly flap, 24.5 m. width, 3.8 m. high, assembled in the workshop

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Hydraulic steel structures for Weirs
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Dam Building

and similar work . . . . .

A BUTTERS CRANE

designed for the purpose

Specially devised for erection in difficult situations the Butters Monotower Crane combines

Great height of lift
Large radius
Full circle coverage
on a
very small erection area

The photograph shows a typical example; this is an electric crane with a maximum capacity of 5 tons and a maximum radius of 100 feet, operating through a full circle.

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2.1m I.D.

Needle valve
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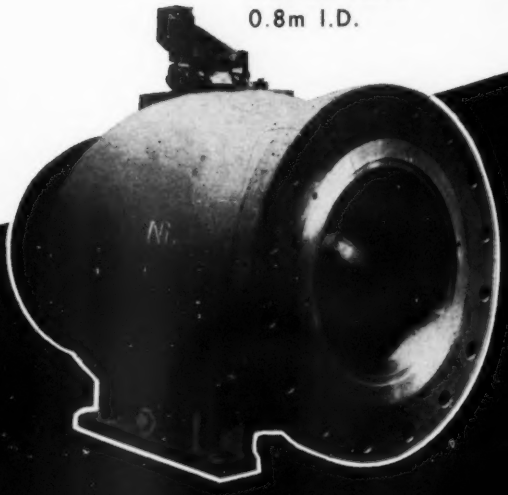

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3.3m I.D.

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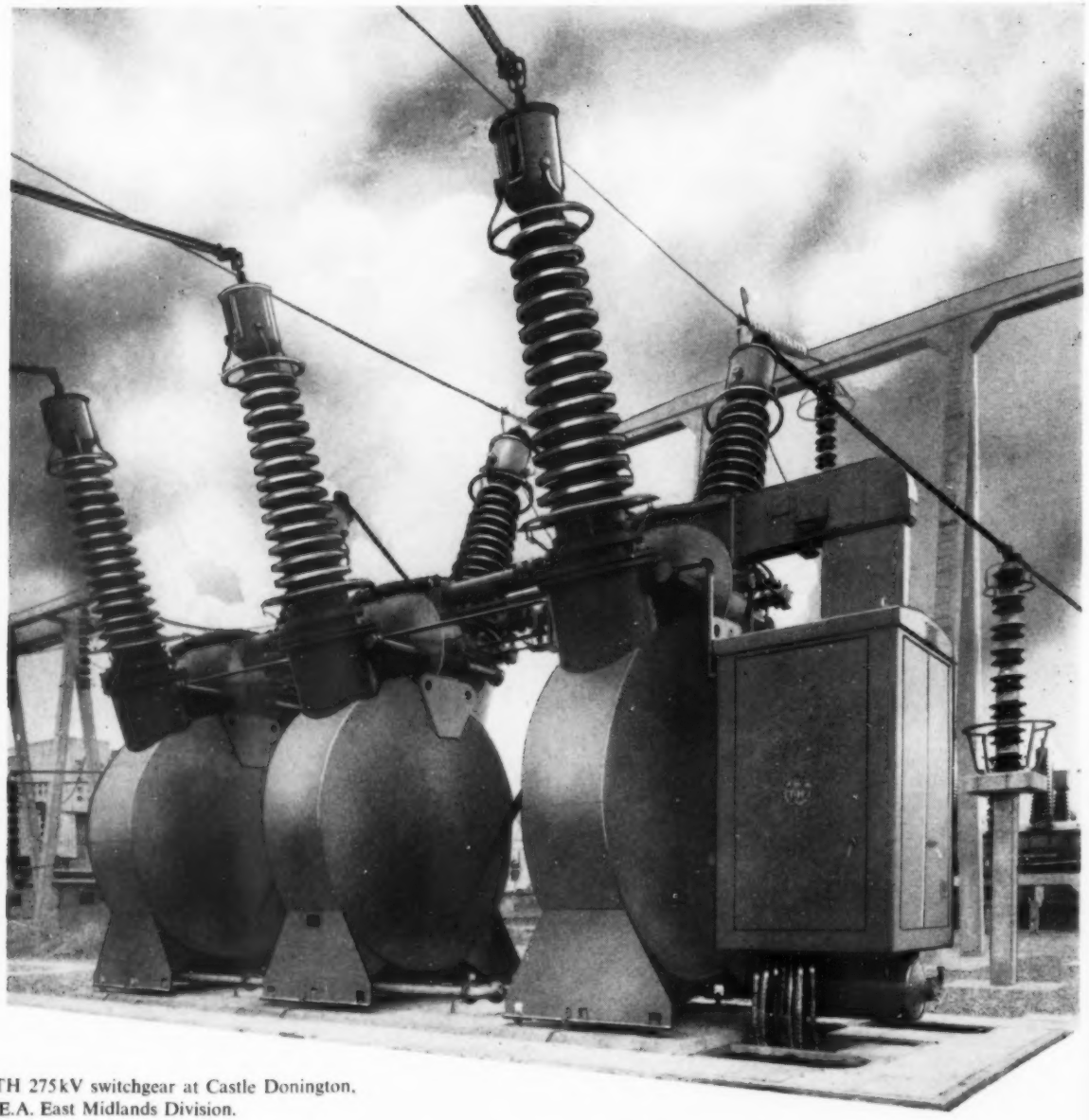

BTH 275kV switchgear at Castle Donington,
C.E.A. East Midlands Division.

B
T-H

330-kV SWITCHGEAR
for
KARIBA

The two million pound order awarded to British Thomson-Houston for the Kariba Hydro-electric Scheme includes 23 oil circuit-breakers of the BTH lenticular type. These units are a direct development of those installed on the 275kV British Super-Grid.

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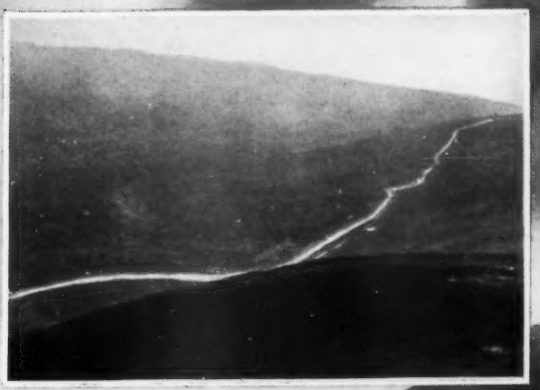
The Contractor shall provide all necessary temporary access...



The problem varies from site to site—here are some 40 miles of access roads built complete with bridges by The Mitchell Construction Company in Glen Almond, Glen Tarken and Glen Lednock for development of the St. Fillans Section of the North of Scotland Hydro-Electric Board's Breadalbane Scheme.

CONSULTING ENGINEERS

Sir M. Macdonald & Partners

THE MITCHELL CONSTRUCTION COMPANY

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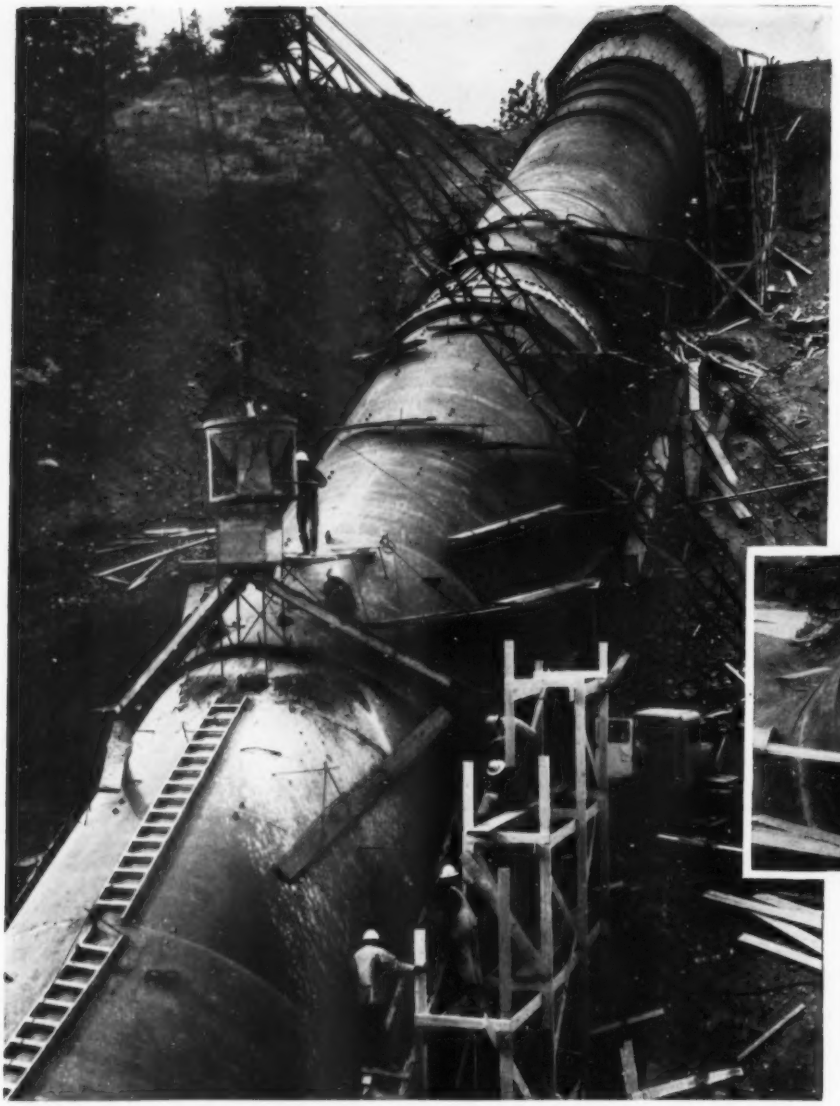

SETON CREEK PENSTOCK

The Seton Creek hydroelectric project is the third major power development in British Columbia in the past few years to use Dresser Penstock Couplings. View above shows workman bolting up the giant coupling.

Coupling at Seton Creek had to have give and take!

At the Seton Creek hydroelectric project in British Columbia, this huge 218″ OD penstock was engineered to flex with the ground, to contract and expand with temperatures varying as much as 120°.

The 3-section Dresser Penstock Coupling was used to flex *with* this movement while providing a bottle-tight joint under a design pressure of 55 psi. (Segmental-type couplings can be designed to withstand much higher heads.)

The middle ring, made of 1¼″ ASTM, A285 steel, was segmental type and field welded at the job site. With this in place, wrenches were the only tools needed to secure the segmental ⅝″ wall cast steel followers. (See photo inset above.)

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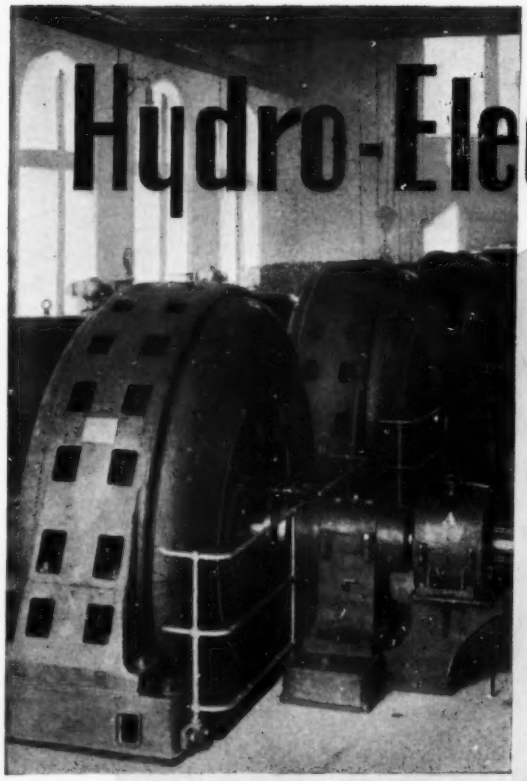
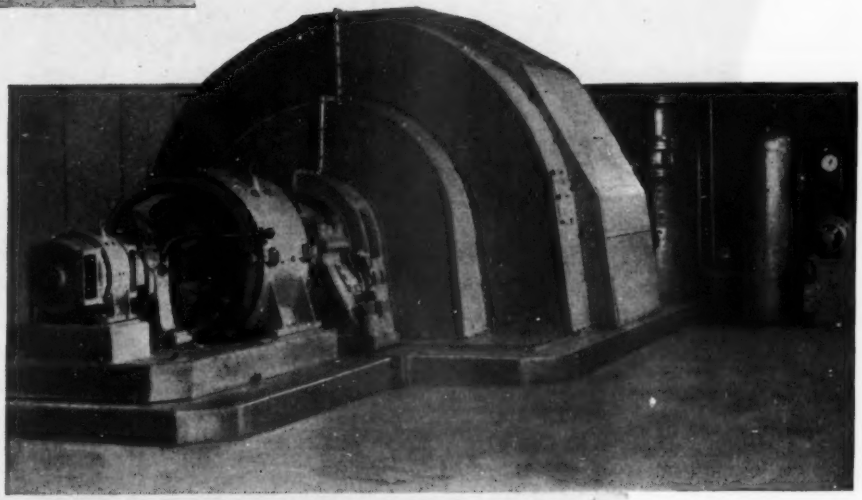


Hydro-Electric Generators

Fifty years ago Bruce Peebles engineered the Snowdon Hydro-Electric Scheme — from tunnelling to transmission lines—the first large project for public supply in Great Britain.

The original Cym Dyli Power Station, pictured at left, is equipped with four 1,500 kVA twin pelton wheel driven generators, which are still running.

These early generators were the fore-runners of the large machines being built to-day for service in many parts of the world.



14,000 kVA hydro-electric generator installed in the Yksnelvane power station, Firdakraft, Norway.

Half a century has elapsed since the first Hydro-electric scheme was completed. The pioneering spirit is still dominant and is revealed to-day in the many contracts undertaken for large hydro-electric plant for many schemes at home and abroad. Ample resources for manufacture and testing enable the largest contracts to be undertaken.

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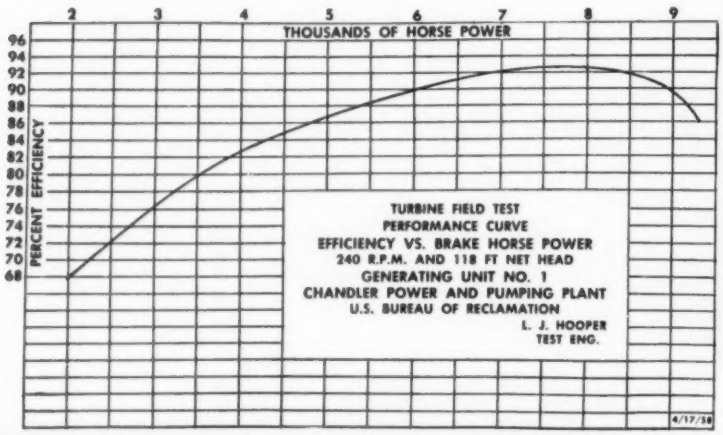

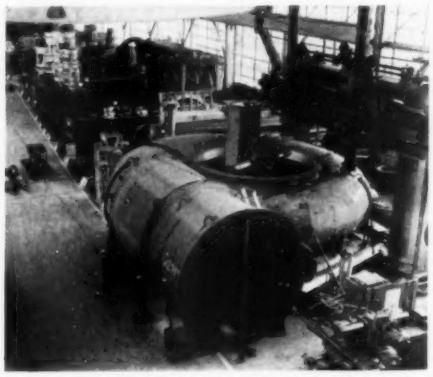

One of the spiral casings for the 8,500 HP turbines on Leffel's 24' boring mill.

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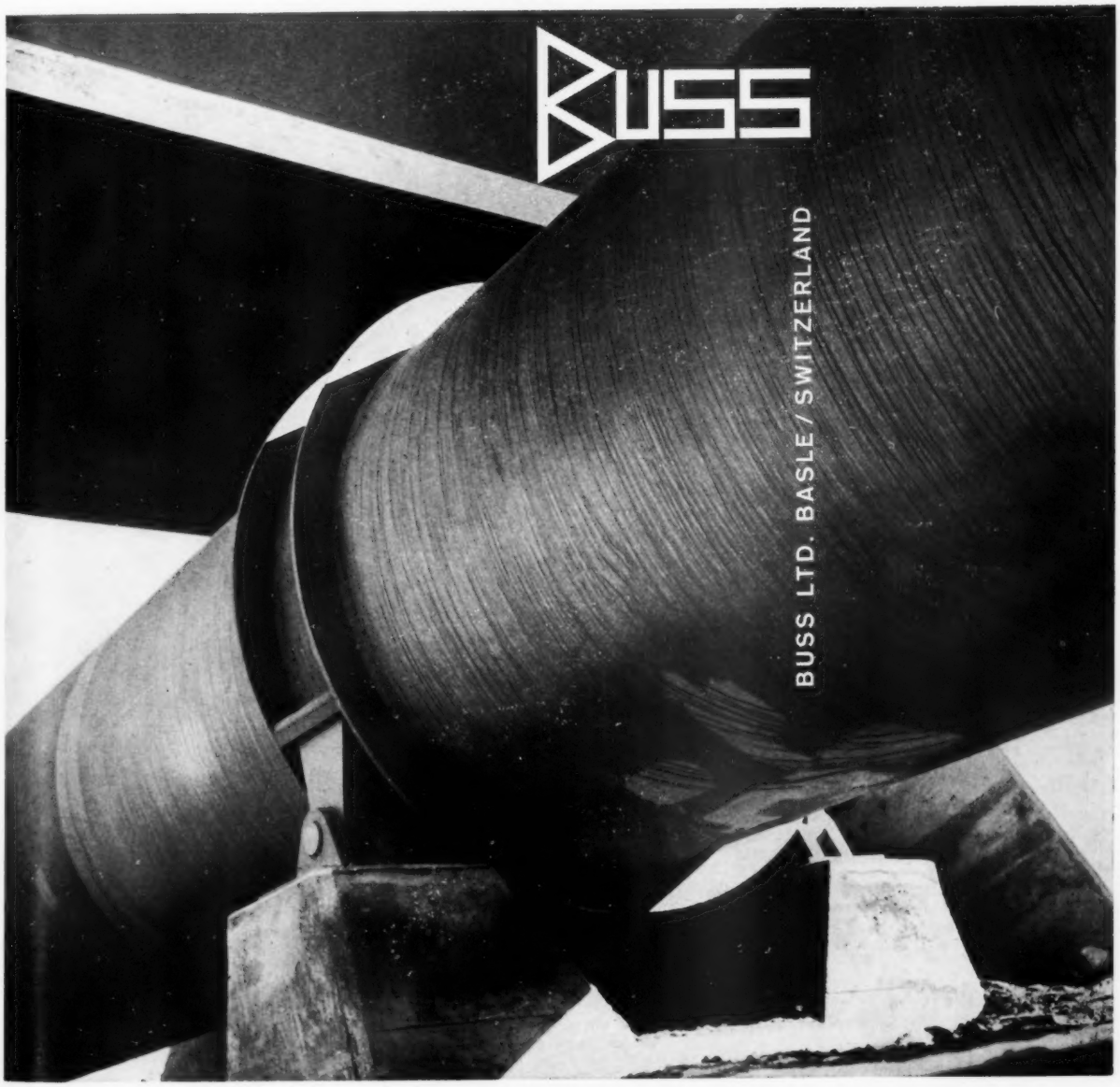

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New power for a Dominion from the Roxburgh Hydro-Electric Project

New Zealand's largest hydro-electrical undertaking, the Roxburgh project, will eventually feed an additional 320,000 kilowatts of electricity into the South Island grid system. The 1,200 ft. Dam, which is the largest ever built in the Dominion, was completed far ahead of schedule by Cubitts' New Zealand Co., in association with Conrad Zschokke Ltd., Geneva, and Downer & Co. Ltd., New Zealand.

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Remote Control

ONE of the most fruitful fields of research in hydro-electric engineering is that devoted to the development of means to operate remote stations and substations by automatic remote-controlled devices. Not only is this development one which brings its own obvious economic saving, but it has also become a pressing necessity for a totally different reason. In many parts of the world the hydro-electric potential now being exploited lies in regions remote from centres of population. Many of the nearer sites are either harnessed or scheduled for development, and it is becoming increasingly difficult to attract engineers and workmen to live in remote regions. The ever-increasing facilities offered by urban life—better educational facilities for children, wider ranges of consumer goods, and other urban amenities—all militate against attracting the skilled and reliable workers of all grades needed for the maintenance and operation of hydro-electric installations.

The fundamental problems of remote-control equipment may be said to have been solved, even in relation to the largest units. Remote starting, synchronising, load control, and shutting down have all been successfully carried out for many years, but in general the remote-control installations now in being are operated over relatively short distances, and in perhaps the majority of cases, by means of pilot wires or by carrier systems using the high-voltage line conductors.

The next step, the use of radio links alone, has been tentatively taken in several parts of the world. The International High-Tension Conference (CIGRE), at its bi-annual meetings in Paris, has devoted considerable attention to this matter. In 1956, for example, there was a paper from the U.S.A. showing that micro-wave systems had been successfully employed over distances of between 50 and 100 miles for the complete control of a 110 kV transmission system. Russian engineers have reported, at CIGRE and elsewhere, reasonably satisfactory experience with radio links, mainly for teletransmission of loading and other quantities, but also for control.

At CIGRE this year there was a most interesting paper from Sweden, surveying progress in that highly developed water-power country in regard to remote control. It showed that complete "automation" of control could effect a saving of 50% in annual operating costs. The problems of evolving radio links for this necessary work are clearly stated. The costs of multiwire systems, carrier systems, and pilot circuits are compared. This comparison is of particular interest because it shows, *inter alia*, that for distances over about 30 miles the radio link is only slightly more costly (in annual expense) than its only rival above that distance, the carrier scheme.

The greatest problem in radio-link operation (as indicated not only in the CIGRE paper mentioned but in many other references in technical literature) is that of fading. But in most highly industrialised countries there is nowadays an incentive to develop reliable long-distance radio links for television. In Great Britain, for example, there are nearly 1,000 miles of ultra-short-wave radio links in daily service to diffuse the two television services. All these must of necessity operate with the high reliability demanded of a public service. It is an ironic comment on the attitude of the public to science that slight failures of a television link, involving the loss for a few minutes of the vision of a crooner's tonsils, would be the subject of a public outcry and funds would instantly be found for duplicating and improving the technical equipment; and because of that attitude, the much more important matter of controlling the hydro-electric schemes on which the whole life of a country (as well as its television service) may depend are given an oblique benefit.

The Swedish paper states, "In comparison with other pieces of equipment in a power system, remote-control equipment is considered to have low reliability." This statement may well be challenged. The radio links, at least, can be made completely reliable,

using unattended booster stations on high ground at intervals of perhaps 20 miles along the route. The relays, selectors, teletransmission meters and other equipment have all been thoroughly tried out in service. Experience with Post-Office equipment, gained under the most arduous traffic conditions, has perhaps not been drawn upon as thoroughly as might be possible.

It seems as if each individual component in a remote-control installation has passed service tests, in other applications, with flying colours. To make the whole as good as its parts, and so capable of being entrusted with complete supervision and control of even the largest installation, all that now seems to be necessary is to institute a greater measure of co-ordination between radio, relay and instrumentation engineers. The result may well mean that the economic status of all hydro plants, *vis à vis* their thermal or nuclear counterparts, is visibly enhanced.

More Power Possibilities in British Columbia

BY arrangement with the Government of the Province of British Columbia the Wenner-Gren BC Development Company undertook to survey a tract of land, virtually uninhabited, between 50,000 and 60,000 miles in extent, and containing the headwaters of the Peace River. The objectives were to assess the hydro-electric potential, the nature and distribution of the forests, mining possibilities and problems of transport. The B.T.H. Export Company acted as technical advisers in regard to the generation, transmission and utilisation of electrical energy, and their preliminary survey disclosed that over two hundred dam sites were available and that the topography lent itself readily to the construction of a storage reservoir 260 miles long which could be utilised not only for firming-up power production from other rivers in the province, but also as an invaluable waterway for the transport of timber, ore and supplies. This reservoir would constitute the largest man-made lake in the world and a preliminary estimate suggests that a potential of four million h.p. could be harnessed between the new lake and the Alberta border. The transmission problem is considered to be promising both from the technical and economic viewpoints. Teams of geologists are now engaged in surface investigations and exploratory drilling.

Power Generation in New Zealand

ELECTRIC power generation in New Zealand for the year ending March 31 was 99·52% of the unrestricted estimate, and an increase of 13·83% over the amount generated in the previous year. In the North Island the increase in the amount generated was 11·63%, and in the South Island the increase was 19·3%. Commenting on these figures, the general manager of the State Hydro-Electric Department, Mr. A. E. Davenport, said it would be apparent that the estimates of future power demand were not excessive and were likely to be realised under conditions of unrestricted supply.

"For 1958, restrictions are necessary in the North Island until additional power stations are available," said Mr. Davenport. "The degree of restriction will depend on the weather, but as the winter closes in, the lack of generating capacity must be felt by all. The degree of restriction which has been set, but not yet achieved, can be regarded as a reasonable minimum. In the week ended March 30, North Island consumption was 63,000,000 kWh—7,000,000 kWh, or 12·5%, in excess of the allocation."

Dominican Republic Dam Contract

THE Dominican Republic Government has announced the awarding of a contract for a £5·4 million hydro-electric scheme at Tavera to the native associated firms of Concretera Dominicana and Mendoza & Armenteros. Ten firms entered for the competition which closed on April 30. These included U.S., Canadian and Italian companies. The Examining Commission stated that their decision was reached after detailed study in which all advantages were weighed, both economic and technical, and the contract finally awarded to the associated firms of Concretera Dominicana and Mendoza & Armenteros as being most favourable to the country's interests. The dam will be built across the river Yaque del Norte in the north-west region of the Republic. It will be a 250 ft. high dam, of the arch type, capable of holding 113,000 acre-ft.

Tiga Rapids Proposal

THE Government of Northern Nigeria is planning a development at the Tiga Rapids, 50 miles south of Kano, on the Kogin Kano river. In an average year the total discharge of this river at the Rapids is of the order of 60,000 million cu. ft., concentrated mainly in the wet season. The project is for a concrete dam with controlled spillway and an associated power station of 10 MW capacity. The reservoir will be upwards of six miles long and more than a mile wide. A pilot dam has been constructed to check the effects of siltation. The cost of the scheme is estimated at £3 million.

Uganda Electricity Board Report

THE Report for 1957 records continuous progress and achievement during the first ten years of the Board's existence. The Board is now selling 21 times as much electricity as it did in 1948 and the number of consumers has increased from 3,263 to 19,819. In 1948 the revenue was £43,000, but last year it exceeded £1,000,000. Generation, which in the early days was from a wood-fired boiler and a diesel station, with a total installed capacity of 2,300 kW, is now almost entirely from the Owen Falls hydro-electric station, the capacity of which has risen during 1957 to 90,000 kW, out of an ultimate total installed capacity of 150,000 kW. During the year, the Board's Consulting Engineers commenced detailed designs for a proposed 180,000 kW hydro-electric station to be erected at Bujagali, 4½ miles downstream from Owen Falls. Investigations have also been continued into possible hydro-electric sites for supply to Fort Portal (Muzizi River) and Gulu, Lira, Masindi, Hoima and Butiaba (Karuma Falls).

Good progress has been made on the Busoga distribution scheme and Kamuli and Kaliro were connected to the Board's mains. Soroti, which had its own diesel generating station, received a supply of hydro-electricity from Owen Falls, following the completion

of the 33,000 V line from Mbale. Work started on the 33,000 V line to supply Mbarara from the Kikagati station and local distribution in Kikagati was completed.

The Board approved other comprehensive schemes to supply large areas of Central Mengo and the Mount Elgon/Pallisa area and consideration was given to schemes for supplying new areas in the Kibuga, near Kampala. The total capital expenditure approved on new schemes was £1,304,000.

Shawinigan's 60th Anniversary

THIS year marks the 60th anniversary of The Shawinigan Water and Power Company which was first granted a charter on January 15, 1898. The first electric generating units with a capacity of 7,500 kW were placed in operation in 1903—a rather far cry from the present generating capacity of 1,284,000 kW, or 1,628,000 kW if purchased power be included. In an address at the annual meeting recently the President, Mr. J. A. Fuller, stated that the Company had every reason to be proud of the way in which it had contributed to the economy of the Province of Quebec. A big part of this contribution, he added, had been the role of the Company's distribution system which had been built up from a nucleus of a number of smaller companies purchased by Shawinigan before 1930. Since that time, and particularly in postwar years, the distribution network had grown substantially. In 1947 it accounted for 29% of firm power revenue, while today, including sales to subsidiary distributing companies and certain municipalities, the figure has grown to 54%. Between 1947 and 1957 the Company built over 5,000 miles of rural lines, at a cost of some $15 million, thus providing electric service to more than 28,000 new customers, mostly farmers. Today almost 42,500 farm customers are supplied by the Company, and rural electrification in the Company's territory is substantially completed.

Electricity Supply in Europe, 1957-1975

A REPORT by the Electricity Committee of The Organisation for European Economic Co-operation (2 rue André-Pascal, Paris 16e) deals with the present condition of electricity supply, the difficulties confronting the medium-term development, and the long-term prospects of the industry. Hydro-power output in OEEC countries is estimated to reach 234 TWh by 1965 and 351 TWh by 1975, that is, to increase 63% during the first ten years and 50% in the following ten. Expressed in terms of capacity, the new plant brought into commission is expected to amount to 14 GW between 1956 and 1960, 14 GW between 1961 and 1965, 15 GW between 1966 and 1970 and 15 GW between 1971 and 1975. Owing to the exhaustion of the more economic water-power sites the cost per kW installed is likely to rise but this does not necessarily put the hydro plant at a disadvantage, *vis à vis* the thermal station, which must inevitably pay more for its fuel as the more profitable seams of coal are exhausted. By 1975 it is estimated that about 25% of the total power requirements of the OEEC countries will be contributed by nuclear plants, 25% by hydro plants and 50% by thermal plants. There is every reason to believe that international exchanges of power will lead to very worth-while economies both in the total amount of power required and in the running costs.

North of Scotland Hydro-Electric Board

THE North of Scotland Hydro-Electric Board's Glenmoriston power station (36,000 kW capacity) is now in full operation, the second 18,000 kW alternator set having completed the commissioning tests. This station is the lowermost installation of the Moriston scheme in Inverness-shire. It is of the underground type and is situated 300 ft. directly below the dam at Dundreggan and its turbines discharge into Loch Ness through a tailrace tunnel four miles in length.

The station will have an estimated average output of 114 million kWh per annum. Generating machinery consists of two vertical Francis-type turbo-alternators operating under a gross head of 306 ft., both sets having been manufactured by the English Electric Company at Harland & Wolff's works at Scotstoun, Glasgow.

The consulting engineers for the scheme are Sir William Halcrow & Partners for the civil engineering, and Messrs. Kennedy & Donkin for the electrical and mechanical work.

The Sasumua Dam

IN a paper presented recently to The Institution of Civil Engineers, Professor Karl Terzaghi of Harvard University gave an interesting account of the design and performance of this dam, which was built to provide part of the water supply of the City of Nairobi. It is an earth-built structure, the major portion of the fill consisting of the products derived from the tropical weathering of basaltic lavas, tuffs and breccias. The clay content of these residual soils is abnormally high and the same can be said of their plastic limits; on the other hand, the dry density at optimum moisture content is abnormally low. The unusual properties of these soils gave rise to doubts as to whether they could be used with safety for the bulk of an earth dam, but a thorough investigation indicated that their abnormal characteristics were assets rather than liabilities.

The paper deals with the geological characteristics of the dam site, the sources of the constructional materials, the reasoning which led to the final design, the performance of the completed structure and the performance of other dams made of similar materials.

New Guinea Rivers as Hydro-power Sources

NEGOTIATIONS are under way between the Queensland Government and the Commonwealth Aluminium Corporation to develop hydro-electric power from the rivers of Papua–New Guinea for processing bauxite from the immense field at Weipa, on Queensland's Cape York Peninsula. The Corporation's agreement with the Queensland Government requires it to begin producing alumina from the bauxite deposits within five years. Investigations by Australian Government engineers have indicated that the rivers of Papua–New Guinea have a potential hydro-electric power of at least 25 million kW.

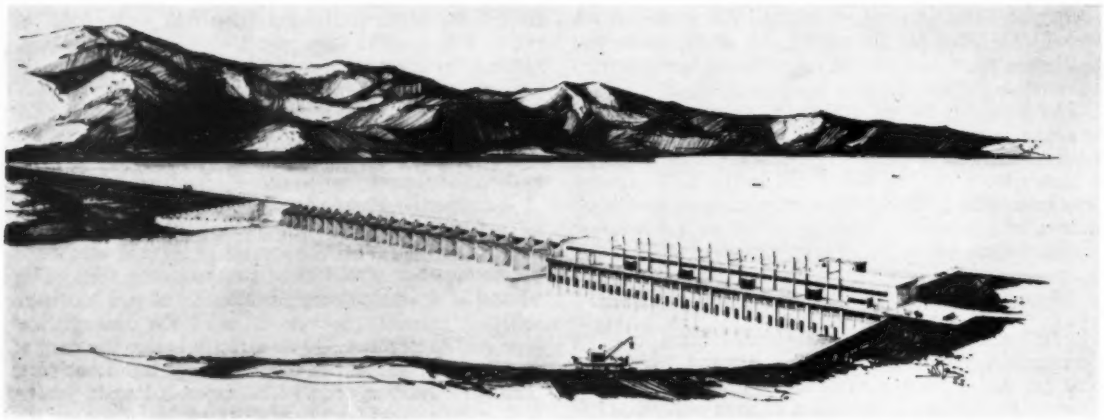


Fig. 1. Drawing of Priest Rapids hydro-electric station, Columbia River, Washington

Priest Rapids Development

The design and contractual requirements of this interesting scheme on the Columbia River are described by George B. Schroyer, head of the Mechanical Department, Harza Engineering Company*

PART ONE

THERE are approximately 150 ft. of undeveloped head on the Columbia River between the Priest Rapids site and the existing upstream Rock Island power plant. The location of Priest Rapids is shown in Fig. 5. The justifiable generating capacity at this time is approximately 1,170,000 kW and it would have been possible to develop this potential with one high dam. Studies that were made, however, indicated the economic advisability of a two-stage plan with one station at Priest Rapids and a second, of approximately equal size, at Wanapum, 18 miles upstream. A further consideration in the selection of a two-stage development was the potential flood hazard to the Hanford Works of the Atomic Energy Commission, in the event of the single high dam being destroyed. Priest Rapids is now under construction and the planning of Wanapum is well under way.

Priest Rapids will be a low-head run-of-river plant. It is of unusual interest because of the size and scope of the financing and construction contract. The quantities of materials involved in the construction are extremely large. According to Mr. Herbert Jenkins of the Grant County Public Utility District No. 2, the 757,000 barrels of cement required would fill a train nearly 25 miles long, and the trucks required to haul the 3,239,000 cu. yards of excavation would extend in a line for 1,150 miles, bumper to bumper. Also, there are many design features which are unusual.

* An early draft of this contribution was presented at a Meeting of the Hydraulic Power Committee, Pennsylvania Electric Association, and is published here with their kind collaboration.

History of the Development

The Priest Rapids Development is sponsored by Washington State's Grant County Public Utility District No. 2. A great part of the early history of Priest Rapids lies under a surface of factional harangues and dissent. While much of the tug-of-war over the scheme was outside of the scope of the Harza Engineering Company's work, the drama inherent therein is, I think, of interest to all who are active in the hydro field.

At the outset, Priest Rapids had the distinction of being one of our country's most coveted dam sites. For some years, it had been scheduled for development by the Army Engineers. The Grant County PUD overcame mounting pressure for a 100% Federal project and secured an alternative approach to the development. This was accomplished by an act of the 83rd Congress which permitted the site to be developed by the District and/or others. Many a disbeliever felt that the Grant County PUD, admittedly small, could never crystallise the financing of a project that, when completed, would cost some $166 million for the Priest Rapids phase alone. Senator Morse publicly doubted that the District could handle a job of such scope, and, in an attempt to head off the bill, called it "a legislative stick of dynamite, which, when the fuse goes off, will do great destructive damage to a Federal power programme of many years' standing."

In the other corner, the Grant County PUD was faced with Governor Langlie of Washington, who wanted the project carried out under the aegis of the

Washington State Power Commission. He dubbed the plan "an unjustified grab of a vast resource."

Spearheaded by Mr. Glenn Smothers, the Grant County PUD manager, and State Senator Nat Washington, counsel to the PUD, Grant County surmounted one legal hurdle after another. It is a great tribute to these men in particular and to the PUD in general, that they finally overcame every objection.

Today, fine co-operation has supplanted the tensions of the past few years. By means of a taxpayers' test suit, the PUD's right to undertake the project was upheld by the State Supreme Court of Washington. Financing, once thought impossible in the hands of a small PUD, was arranged by the District's contracting in advance for the sale of the power to a group of public and private utilities. With markets in the power-thirsty north-west guaranteed, the PUD issued tax exempt bonds. Their lower yields in comparison to taxable bonds of course reduce the heavy interest burden borne by the development.

For its part, the Federal Government will underwrite the future navigation features of the development and will reimburse the District for lost revenue resulting from a draw-down in pond level in anticipation of a major flood. And so, a small county and a group of utilities have joined together for the construction of a project which is large even for the Pacific north-west.

Fig. 2. Hub-arm assembly for one of the turbine generator sets in maker's works

Fig. 3. Semi-finished shaft for one of the turbine sets in the English Electric Company's works

Financing of the Development

District. A public utility district is a body unique to the State of Washington. The districts were created by an enabling act of the State and are duly established political sub-divisions and municipal corporations of the State. They are authorised to own and operate generating facilities and to distribute power to the consumer, both public and private. They are legally able to enter into power contracts, within or without the district, to issue bonds, and to establish rates adequate to provide sufficient revenue for the payment of principal and interest on the bonds and to cover all operating, maintenance, renewal, and replacement expenses. They are, however, non-profit organisations and the bond interest is exempt from Federal taxes.

At present, Grant County Public Utility District No. 2 has no generating facilities and must buy approximately 260 million kWh of energy annually from the Bonneville Power Administration. The distribution system of the District serves nearly all of the county except two small towns which are served directly from Grand Coulee. The population of the service area is approximately 37,000 persons. The District has more than 13,000 customers, including some 800 irrigation customers, in the Columbia Basin project. This project, in the Big Bend country of Central Washington, will ultimately include over half a million acres of irrigated land within Grant County. The land will receive its water from the Grand Coulee pumping plant. It represents a sizable future electrical load. The anticipated energy requirement of the District in 1965 is approximately 1,000 million kWh with a maximum

load of 222 MW. The area is one of extremely rapid growth, the population of Grant County having increased 65% between 1940 and 1950 and another 62% between 1950 and 1955. Electrical consumption reached the staggering annual figure of 12,669 kWh per domestic customer in 1955 at an average cost of only 0·93 cents per kWh. This compares with the national average of 2,751 kWh per domestic customer at a cost of 2·64 cents per kWh.

The present commissioners of the District are F. W. Ault, president; William Schempp, secretary, and George Schuster. The District manager is Glenn A. Smothers.

Power Contracts. The District has entered into power sales contracts with several utilities for the sale of 63·5% of the 630,800 rated kW output of the development. The District will reserve the remaining 36·5% or 230,242 kW for its own use. These several utilities will, in monthly instalments, pay their proportionate share of all the costs, including principal and interest on the bonds, insurance, operation, maintenance, renewal and replacement expenses; and on the same proportionate basis, they will share in peaking power, energy production, and the benefits of pondage. The power sales contracts are binding until the year 2005. This type of power contract is unique in that the purchasers agree to pay the annual costs for the power rather than a fixed price per kW or kWh.

The two large power purchasers are the Pacific Power and Light Company and the Portland General Electric Company, each with commitments for 13·9% of the plant output. The cities of Seattle and Tacoma are each purchasing 8·0% of the output and the Washington Water Power Corporation will purchase 6·1% of the output. Smaller allotments of power will go to the PUDs No. 1 of Cowlitz County and Kittitas County, and to the cities of Eugene, Forest Grove, McMinnville, and Milton-Freewater, all in Oregon.

The purchasers, through an advisory committee, will have a direct voice in the operation and management of the development and in the scheduling of the delivery of the power, subject, of course, to available stream flow and to the operation of the Northwest Power Pool. After the plant has begun operation, the District is obliged to employ an engineering firm which will report on the management, operation, and maintenance of the development.

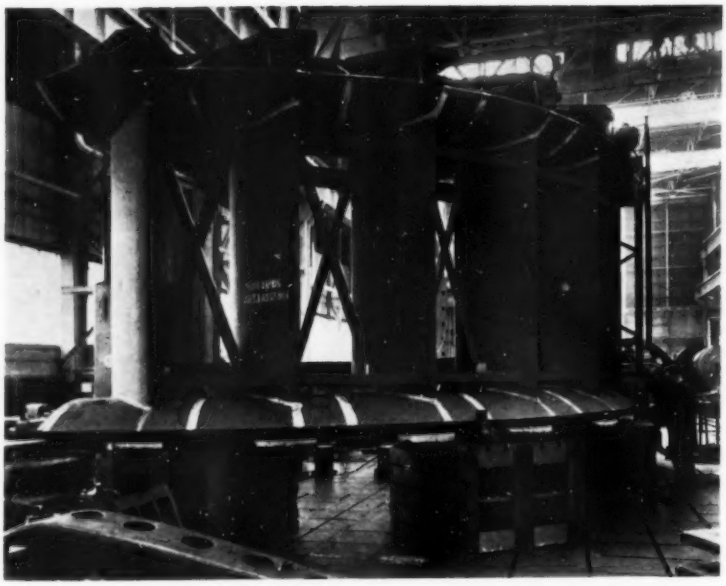


Fig. 4. Face finishing a stayring quadrant

TABLE I.—CAPITAL COST

Direct construction costs, including engineering and approximately 15% reserve for contingencies		$124,900,000
Working capital		$3,000,000
Deposit to Reserve Account in the Bond Fund		$6,432,500
Bond discount and other financing expenses		$7,074,962
Interest on Bonds during construction (1,900 days) ...	$33,484,379	
Less: Estimated interest earned during construction	$8,891,841	
Net interest during construction		$24,592,538
Total		$166,000,000

Bond Issue. The development was financed by a bond issue of $166 million, bought on July 9, 1956, in a negotiated sale by a nation-wide banking syndicate headed by Halsey, Stuart & Co., Inc., John Nuveen and Co., B. J. Van Ingen & Co. and Blyth & Co., Inc. The bonds were bought at 96·2 and reoffered at 99, and will pay 3 7/8% interest, free of federal income tax. Only a portion of this issue represents the anticipated construction cost of the project as is shown in Table I.

The bonds are redeemable at the option of the District and for this purpose a sinking fund is provided in the power purchase contracts.

Power Costs. Based on the nameplate rating of 630,800 kW, and the direct construction cost of $124,900,000, the cost per kW of the development will be $198. On the basis of the total bond issue, the cost would be $263 per kW. This would indicate average energy costs of approximately 2·2 mills per kWh for the expected annual energy production of 4,200 million kWh. This is inexpensive power, even for the Pacific Northwest.

Construction Contract

General. Competitive bids for the construction of the development were received on March 12, 1956, and an award made to the low bidder, Merritt-Chapman & Scott Corporation, on July 9, 1956, although a letter of intent had been issued on May 14. The contractor expects to complete construction in the summer of 1960, nearly a full year ahead of the specified completion date.

Contract Stipulations. The construction contract is of the unit price type; i.e., quantity estimates of various items, such as pounds of reinforcing steel, are given in the specifications and the bidder quotes a unit price for furnishing and installing these items.

Such a type of contract permits construction to proceed concurrently with the preparation of detailed construction drawings by the engineer. Naturally, many items that could be adequately studied in the relatively short amount of time available for engineering, were specified on a lump sum basis. Such items as diversion and care of water are in this category.

There are several unusual features about the Priest Rapids contract, foremost of which is the inclusion of major items of mechanical and electrical equipment in the general construction contract. The turbines, governors, generators, transformers, generator switchgear, cranes, gates, elevators, and certain items of the miscellaneous mechanical and electrical auxiliary equipment are purchased by the general contractor in strict conformity with rigid specifications. To our knowledge, the only other project wherein a similar purchase system was used, aside from "turn-key" jobs, was the Tri-Dam Project in California. Approximately 94% of the estimated total cost of the construction work, equipment supply and installation is included in this one contract. The major items of equipment not included in this contract are represented by the fish-pass facilities, which had not been finalised at the time of writing the specifications, and include the fish pumps, pump-house crane and the weirs, gates and valves for the fish ladders. Reservoir relocation and the transmission lines are not included in the construction contract. Certain of the smaller items of power-house equipment, such as the sump pumps, air conditioning and ventilation equipment and certain switchgear and electrical auxiliary equipment, likewise were not included. Such a form of contract, it is hoped, will have decided advantages for the District. Since responsibility for so much of the development is centralised in the hands of one contractor, it is felt that the District will be relieved of the problems of scheduling deliveries, co-ordinating the efforts of manufacturers and mediating between the possibly conflicting interests of subsidiary contractors. One contractor only is held responsible for delays, equipment performance, and quality of construction. The bidders were required to state in their proposals the names of their proposed major equipment sub-contractors, and no deviation from this list is acceptable after contract award, except with District approval. The bidders could only solicit quotations for major permanent equipment from a list of manufacturers previously approved by the District.

A liberal construction period of 1,900 days was specified with a penalty of $3,000 per day per turbine-generator unit for late completion. A bonus of $1,300 per day per unit is provided for early completion. On this basis, the contractor may earn a daily bonus of $10,400 for the full eight units or he may incur a penalty of $24,000 per day. The daily interest on the bonds is $17,868, which gives the District protection for either contingency. The only recognised *force majeure* provisions are those attributable to earthquake or acts of war.

The District is protected by the contractor's performance bond in an amount equal to 50% of the value of the Contract. Approximately 10% of all monthly payments will be withheld from the contractor until all work is completed, and accepted by the engineer. A further total of one million dollars will

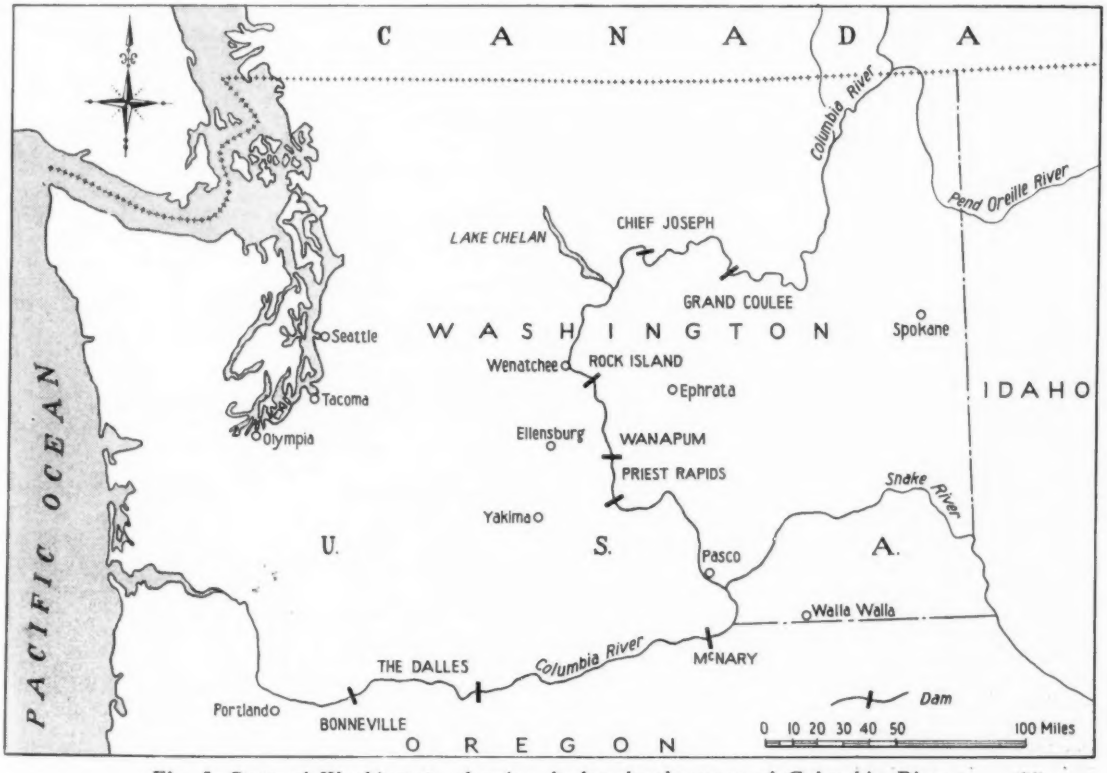


Fig. 5. State of Washington, showing hydro development of Columbia River

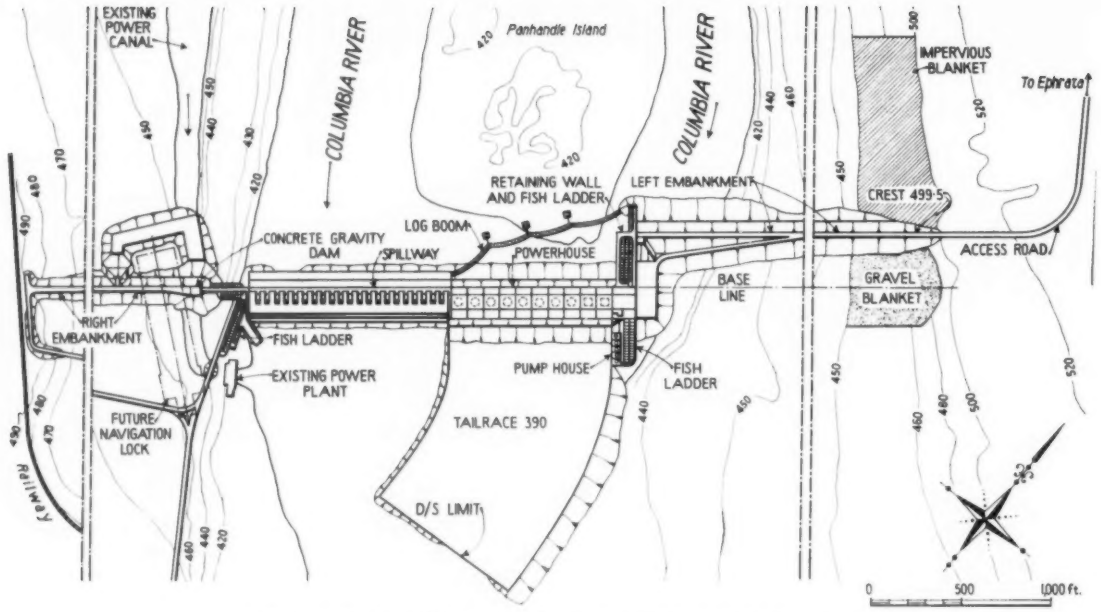


Fig. 6. Plan of Priest Rapids works on Columbia River

be withheld until the completion of one full year of satisfactory operation.

Bids Received. Five bids were received for the general construction contract and all bids, except that of Merritt-Chapman & Scott Corporation, represented combines of most of the large American general contractors. A summary of the bids received is shown in Table II.

TABLE II.—BIDS RECEIVED

	Total amount of proposal
Merritt-Chapman & Scott Corporation ...	$91.880.625
The Henry J. Kaiser Company and Associates	$93.160.204
The American Pipe & Construction Company and Associates	$97.494.814
Grant County Constructors	$103.996.562
J.T.M. Constructors	$108.889.162

All of the bidders had some permanent components of foreign manufacturer in their proposals excepting Grant County Constructors, who based their bid on equipment of domestic manufacture throughout. The American Pipe proposal, however, included an alternative bid, at a slight price increase, for domestic transformers. Although advance copies of the specifications were sent to foreign manufacturers of governors, cranes, gates, trashracks, switchgear and bus structures, the bidding, throughout the proposals of the general contractors, indicated that foreign equipment was restricted to the generators, transformers, and one proposal for the turbines.

The engineer's estimate for the contract was $125 million, some 15% higher than the highest bid. The range of the bids is surprisingly small for a project of this magnitude, indicating, in our opinion, that all the contractors understood the requirements of the specifications.

It is difficult to explain the extremely low value of the bids in comparison with the engineer's estimate. It is probable that foreign competition on the major equipment items tended to reduce prices, although several of the bidders quoted turbine prices for American machinery which were lower than the bid of the successful contractor who quoted on foreign turbines. In any event, the successful bidder's turbine prices were $7 million and the generator prices $10 million under the engineer's estimate which had been based on preliminary price quotations from American manufacturers. The engineer's estimate contained escalation; however, the bid prices of Merritt-Chapman & Scott were firm.

The large number of bids received for a project of this magnitude indicate keen contractor competition and this probably tended to reduce the bid prices.

The bonus and penalty clauses were also effective in lowering the bid prices; the specified completion time of 1,900 days is liberal and the three low bidders proposed completing the development approximately one year ahead of schedule, thus becoming eligible for a bonus of nearly four million dollars.

Description of the Development

General. The Priest Rapids Development, shown in Fig. 6, is a "run of river" hydro-electric plant consisting of an integral intake and power house with an initial installation of eight Kaplan turbines each rated at 114,000 h.p. under a net effective head of 78 ft. The turbines are directly connected to generators rated 83,000 kVA at 0·95 power factor. Adjacent to the power house is a gated spillway containing 22 tainter gates. The spillway connects through a concrete gravity non-overflow section to a free-draining earth-fill embankment extending to high ground at the abutment on the right bank. The left embankment is of similar earth-fill construction. The overall length of these structures is approximately 8,400 ft. Extensive provisions are made for the upstream passage of migratory fish and consist of a fish ladder at each bank of the river and a system of pumps to supply supple-

mental attraction water for the ladders. Initial provision has been made for the future construction of a navigation lock. There will be no substation at the project and the two outgoing 230 kV transmission lines will connect the project to the Bonneville system at its Midway substation.

A natural drop in rock elevation on the left side of the river favoured the erection of the power house and tailrace in this area.

On the other hand, the higher rock surfaces existing on the other side of the river made it mandatory to place the spillway, with its shallower excavation, on the right bank.

Site. The Priest Rapids Development is on the Columbia River in the semi-desert area of south central Washington. The site is located approximately 200 miles south-east of Seattle and 55 miles north-east of Yakima.

The area is subject to great extremes in temperature with winter lows of from 10° to 20°F. below zero and summer highs which are frequently above 110°F. The area is subject to violent dust storms and in the fall of 1956 there were several storms with wind velocities of from 80 to 100 m.p.h. The site is located 397 miles from the river mouth, above the confluence of the Columbia and Snake Rivers, and is approximately 100 miles upstream from the McNary Project of the U.S. Army Corps of Engineers. The nearest existing upstream development is the Rock Island Project of the Chelan County Public Utility District, 56 miles distant. Priest Rapids is approximately 200 miles downstream from the Grand Coulee dam. The location of the development with respect to other main stem projects on the Columbia is shown in the profile diagram, Fig. 7.

The drainage area of the Columbia River above the site is approximately 95,500 square miles and the average annual flow is 120,000 cusecs or an equivalent run-off of about 17·5 in. of water over the basin. The maximum recorded flow at the site is 740,000 cusecs and the minimum recorded flow is 21,000 cusecs. Peak flows normally occur in May or June and result from the melting of the snow pack on the mountains. Conversely, low flows occur in the winter months when the snow is accumulating. The natural flow of the Columbia now is appreciably modified by the regulation afforded by such upstream storage reservoirs as Grand Coulee, Albeni Falls, Hungry Horse, and by the Flathead and Kootenai Lakes. The minimum permissible flow as stipulated by the Atomic Energy Commission at the Hanford Works is 36,000 cusecs.

The Priest Rapids site is typical of a large portion of the Columbia Basin in that the area is underlain by a thick succession of basalt lava flows. In the neighbourhood of the site, the flows have been folded into a series of anticlines with east-west trends which rise 1,000 to 3,000 ft. above the general level of the plateau. Above the mouth of the Snake River, the bed of the Columbia is largely filled with alluvial deposits up to depths of 300 ft. or more. The Priest Rapids site is one of the few locations where the basaltic bedrock is exposed in the river bed. This rock is deeply channelled so that islands are commonly exposed in periods of low flow. Priest Rapids, from which the project gets its name, is a series of rapids approximately eleven miles long with an average gradient of 6·5 ft. per mile.

Power house. The power house, shown in Fig. 8, is of the indoor type and is integral with the intake. Although a cost saving of approximately $7,000 per unit would have been realised by designing the plant as a semi-outdoor structure, the decision in favour of an indoor plant was made on the basis of such intangible factors as greater protection from weather during the erection period and when the units required to be dismantled, greater storage space, increased flexibility of crane operation and crane hook coverage. A full outdoor plant was not considered because there was a possibility of the generators being flooded, the generator thrust bearing bracket sole plates being at El. 421·5 and maximum design tailwater at El. 462.

Initially, the power house will contain eight generating units, two skeleton units for future installation and an erection bay. Additional units when required will be added east of the erection bay. The intakes and spiral case of each unit will be of reinforced concrete construction, and the draft tubes will be of the elbow type of reinforced concrete. The power house will have a length of 1,075 ft, a base width of approximately 197 ft. and a generator hall width of 70 ft. The superstructure will be formed by the intake wall and by a reinforced concrete downstream wall. The roof will extend over the two skeleton units and will be constructed of structural steel covered with insulation and built-up roofing. Transformers will be

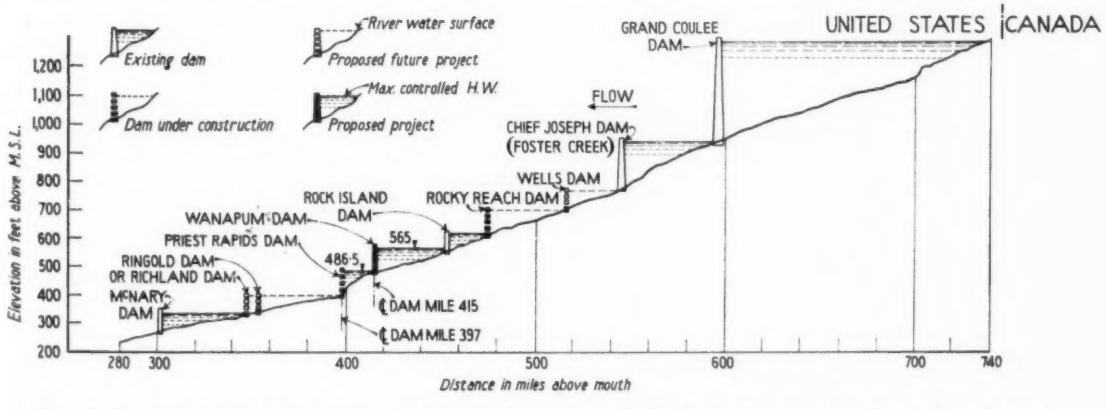


Fig. 7. Profile of Columbia River indicating dams already built, under construction and projected

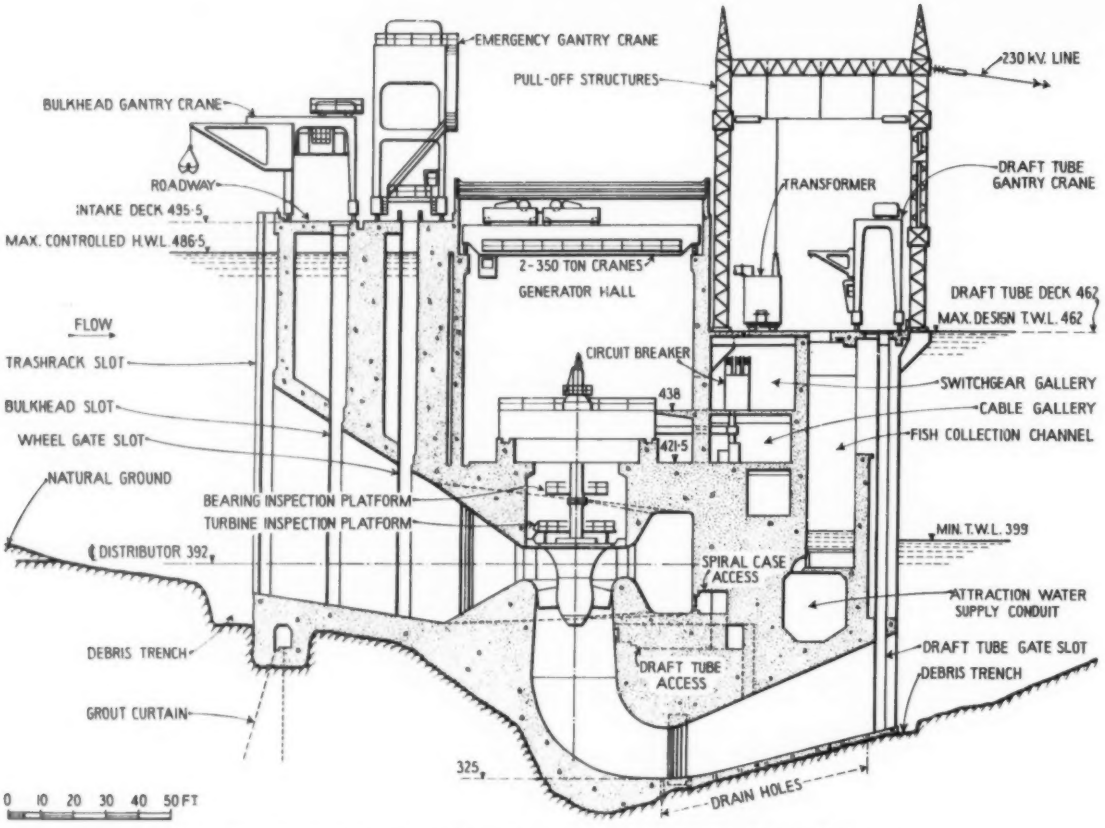


Fig. 8. Transverse section through power station, Priest Rapids

located on the draft-tube deck and the control room will be located between units Nos. 8 and 9 at the generator balcony floor level.

The erection bay monolith will be located east of the generator room and will house the erection space as well as mechanical and electrical auxiliary equipment. The erection floor will be at the generator balcony level, with an unloading platform at El. 462.

Trashracks provided for each intake passage will have an 8-in. bar spacing and will be designed for a velocity of approximately 3·5 ft. per sec. through the gross opening. A trash rake attached to the bulkhead gantry crane will clean the racks and deposit the trash in a dump truck which will run between the gantry rails. Larger trash, such as trees, gravel and waterlogged debris which gathers in a trench upstream from the intake, will be handled by a clam-shell bucket operated from a jib on the bulkhead gantry. The 25 ton bulkhead gantry, in addition to handling the trashrack sections, will also handle the intake bulkhead gates. These gates will be used in the event of major unit outages or for repairs to the downstream emergency gate guides. Bulkhead gates are provided for each of the three intake passages of any two turbines. In addition, two further sets of bulkhead gates will be used to block the intake of the two skeleton units.

The method of closure of the turbine intakes in the event of a runaway, is most unusual. The method employed at many projects, including the McNary and Dalles installations, consists of three hydraulically operated gates for each unit, controlled from a central pumping unit. The method adopted for Priest Rapids consists of a single gantry crane, shown in Fig. 9, which will transport, lower and raise all three gates simultaneously for any intake. The gates will be permanently attached to three separate hoists on the gantry and will be of the wheeled type, 42 ft. 6 in. high by 20 ft. wide. The total load from each gate during lowering under a turbine runaway condition is estimated to be 225 tons, making the total crane capacity 675 tons. This will be one of the largest gantry cranes in point of lifting capacity ever constructed. When flow has been stopped by the wheeled gates, a set of bulkhead gates will be lowered into the upstream slots by the bulkhead gantry. The wheeled gates then will be raised and held in reserve for use at another unit. Although the crane will be controlled from the cab, and is not under governor overspeed switch control, it is extremely rapid in action for such a large structure, having a travel speed of 125 ft. per min. and a lowering speed of 12·5 ft. per min.

The estimated saving procured by this arrangement of gantry and gates, compared with individual gates and hydraulic cylinders for each of the intake passages is $2,105,000. To our knowledge, such a crane has never been used in this country; however, a somewhat similar but smaller crane is operating successfully at the Donzere-Mondragon plant of the Compagnie

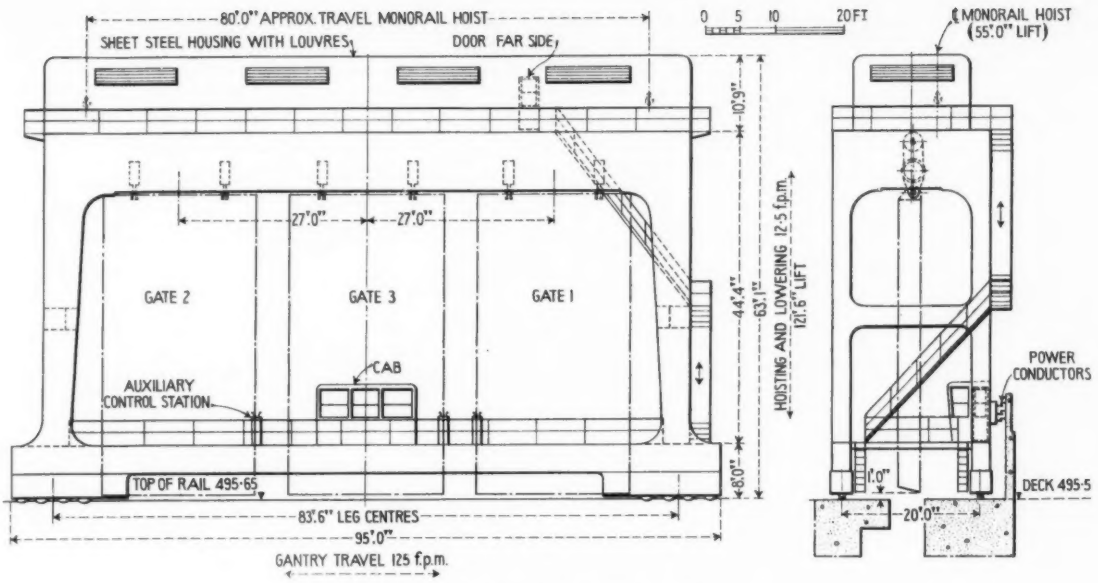


Fig. 9. Emergency gantry crane

Nationale Du Rhone in France, although the gates are articulated and the crane operates either on governor control or by push button from the control room. Also, a similar crane is in use at the Ottmarsheim Power Station of the French Office of Power Stations, and like Priest Rapids, operates under cab control only. Both the bulkhead crane and the emergency gantry crane and its gates are being manufactured by the Pacific Coast Engineering Company.

The turbines will be of the vertical-shaft, adjustable-blade propeller (Kaplan) type and will have a rating of 114,000 h.p. under a net head of 78 ft., which corresponds to 105% of the rated generator load at 0·95 power factor. These will be the largest Kaplan units in physical size on the Columbia River, having a runner diameter of 284 in., being exceeded in size in the United States only by the 292-in. Pickwick units. The speed of rotation is 85·7 revs per min. The English Electric Company, after contract award, proposed several modifications in the specified turbine design which are common in Europe, and which will result in a rebate to the Grant County PUD of approximately $415,000. Specifically, the turbines will include the blade servomotor in the runner hub instead of in the turbine shaft; the turbine guide bearing will be self-lubricating instead of pressure lubricated; the shaft seals will be of the carbon ring type rather than hydraulic packing; and the wicket gate stem seals will consist of neoprene sealing rings rather than hydraulic packing. Governors will be of the single-cabinet actuator, oil-pressure type manufactured by the Pelton Water Wheel Company.

The generators, also of English Electric manufacture, will be rated 83,000 kVA, 0·95 power factor, 3-phase, 60-cycle at 13·8 kV. They will be of the umbrella type and will be provided with a direct-connected exciter operating in conjunction with a static-amplifier type voltage-regulating equipment. The generators will be of the totally enclosed type provided with air-to-water heat exchangers; however, a system of automatically controlled louvres in the air housing and automatic throttling valves in the cooling-water supply system will permit the waste heat to be utilised for room heating purposes. A price reduction of approximately $312,000 will accrue to the District for the acceptance of generator shafts with a portion of the thrust collar attached to the shaft by a shrink fit. Integral thrust collars had been specified; however, the large size of the forgings required proved to be beyond the assured capability of European suppliers. Each pair of generators will be connected to a power transformer through station-type cubicle switchgear by means of isolated phase bus. The switchgear will be made by the General Electric Company. Generator circuit breakers will be of the compressed air type. The transformers, of English Electric manufacture, will be rated at 183,000 kVA, 13·2–230 kV and will be of the 3-phase FOW type. Each pair of transformers will be paralleled on the 230 kV side through disconnect switches located on the pull-off structure on the draft-tube deck.

Two indoor type power house bridge cranes will be connected together by a system of three lifting beams when handling the generator rotor, which is the heaviest lift. Each crane will have a rated capacity of 350 tons and will be provided with two trolleys, each trolley being equipped with a main hoist of 175 tons capacity and an auxiliary hoist of 30 tons capacity. The cranes are being manufactured by the Moffett Engineering Company.

Gates will be provided for closing the draft tubes of two units. The draft-tube gates will be of the vertical-lift, sliding type, of all-welded steel construction. Each gate will be approximately 21 ft. wide and 27 ft. high and will weigh approximately 35 tons. The gates will be stored in gate slots below the draft-tube deck and will be transported by means of a gantry crane which also will service the power house fish-collection channel.

(To be continued)

Bottom Discharge Valves for Dams

Measurements of the motive force and negative pressures, and their application from a model to the actual design, are discussed by Bohuslav Matocha, M.Sc. (Czechoslovakia), CKD-Blansko

FOR the discharge of excess water from dams and, if necessary, for emptying reservoirs for repairs, various outlets are provided. They are arranged at the deepest point and are often used to bypass water while the dam is being built.

Each outlet is provided with at least two valves, and sometimes, for the sake of absolute safety, an emergency valve may also be fitted. This valve is usually held in its fully opened position, but is arranged for quick closing to meet a sudden emergency. The other valves are generally of the sluice type or, if the outlet is small, a wedge-shaped slide valve may be used.

In designing the operating valve the aim is to attain the best possible mechanical and hydraulic properties, not only in the fully open position, but also in all the intermediate positions. The requirements from the mechanical point of view are simplicity of design, safe and reliable operation, minimum of attendance and low wear. From the hydraulic point of view a smooth and undisturbed flow is demanded, leading to low pressure loss in all positions of the valve, high outflow coefficient, elimination of cavitation phenomena and consequent vibration, good energy dispersal characteristics, and watertightness.

These somewhat conflicting requirements are fulfilled in differing degrees by the various valves in common use. For instance, the roller valve shown diagrammatically in Fig. 1 affords an undisturbed flow of water in its fully open position so that the pressure loss in this position is almost zero. The closing elements used in this case are two parallel rollers mounted in semicircular housings. In their open position the rollers provide a clear cross section which is identical with that of the pipeline; in a partly open position, however, the flow conditions are less favourable, as the slit-shaped passage which is formed causes separation of the jet from the pipe to take place around the entire circumference.

More favourable results in partly open positions are afforded by the Johnson valve, the original design of which is shown in Fig. 2. Movement of the operating needle is achieved by the alternate admission of pressure water into space *a* during opening, or *b* during closing.

An improvement of the original design is shown

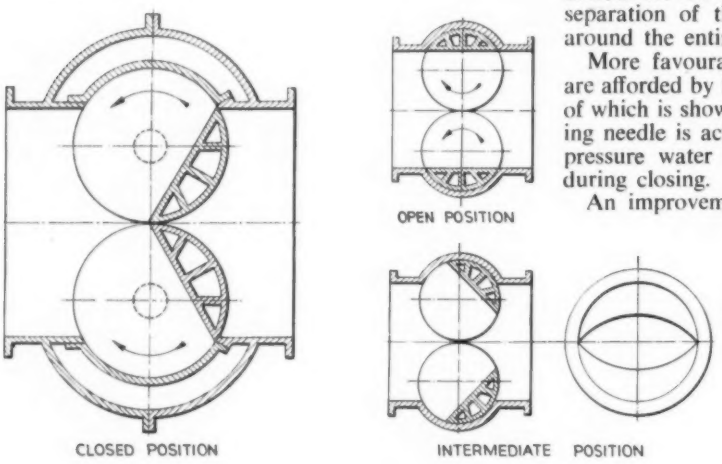
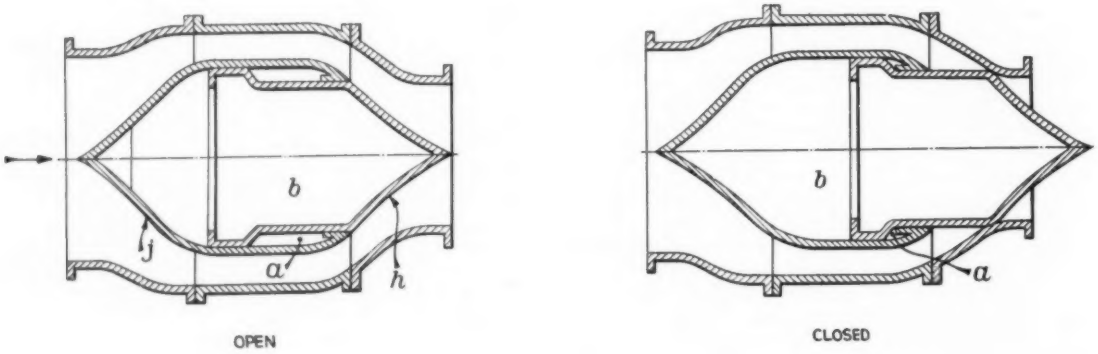


Fig. 1

Fig. 2



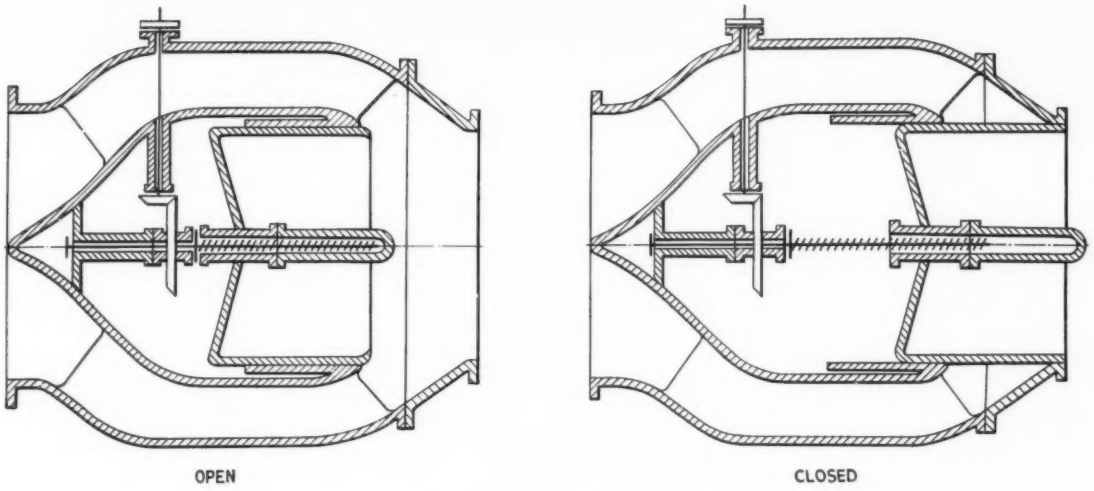


Fig. 3

diagrammatically in Fig. 3. Here, the moving head is replaced by a cylinder which moves inside a fixed core extended upstream. The valve is closed mechanically by means of gears. On the whole, this valve has good flow characteristics, but is unsatisfactory at openings below 35%, at which the discharge jet becomes unstable and pulsates.

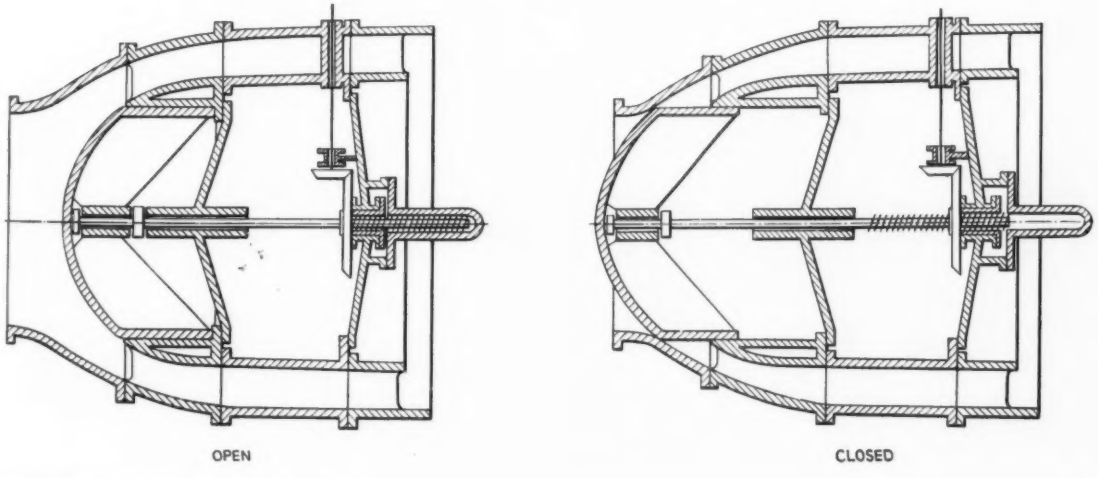
A further improvement of the Johnson valve is shown in Fig. 4 and represents a design which is fitted in most of the high dams built in Czechoslovakia. It differs from other types in that it closes in the direction opposite to that of the flowing water. It consists of two main parts:

1. A bush which surrounds the core of the valve and has a cross section widened circumferentially in comparison with the cross section of the piping controlled by the valve.

2. The core proper of the valve which consists of a fixed part and a moving head or piston which is sealed against the fixed part by a circular seal.

Both these main parts of the valve are joined together by hollow streamlined radial ribs which serve to admit air into the discharge jet and to accommodate the shaft of the operating mechanism. The operation of the valve is effected by an electric motor driving a train of gears, as shown in Fig. 3. The water jet issuing from the valve has the shape of a slender hollow cone with ample access of air to the hollow space. This facilitates the destruction of energy without a large amount of spray and relieves the hydraulic load on the stilling basin. Another advantage is that the bush behind the seating surface is never exposed to the full water pressure and can therefore be of lighter design. There is also the important feature that the water pressure against the piston can be balanced to a large extent by admitting pressure water to the inner channel behind the piston through holes arranged in the face of the piston. Since the pressure against the face of the piston, which the valve is discharging, varies from its centre toward its circumference, the position of these equalising holes may even govern the direction of the resultant force acting upon the piston. For the sake of economy in the operating mechanism, the positions of the equalising holes are

Fig. 4



chosen so that a minimum force acts upon the piston. This is best done by measurements on a model, as will be described below.

As already mentioned, this valve, known also as the hollow-jet Johnson valve, has the advantage that the shape of the discharge jet greatly facilitates the destruction of energy of the flow of water. Nevertheless, the flow conditions are not ideal. Compared with other valves, the capacity of the Johnson valve at full opening is not so favourable, for the core of the valve disturbs the smooth flow of water. The flow changes its direction suddenly and hits the piston wall and the ribs of the circular space, causing increased friction with the walls and consequent higher loss of pressure. The incoming flow of water first hits the convex surface of the piston, which causes it to spread over the entire circumference into the annular passage, which decreases in size in the direction of the flow and in which joining ribs are arranged. At these points of sudden transition, negative pressures (with respect to atmospherics) are produced which create conditions favourable to cavitation.

Thus there are critical points for the occurrence of cavitation, at the inlet in front of the piston, where there is a sudden change of flow, in the annular space near the ribs, and at the point of contact of the moving and fixed parts of the core where the flow of water separates from the entire circumference of the core when the valve is in an intermediate position. To ensure smooth and safe operation of the valve, these points must be sufficiently supplied with air, and for this reason the valve is fitted at the end of the outlet piping on the air side of the dam. This reduces the risk of cavitation and the corrosive influences connected with it. The required cross sections for air admitted to the critical points are best determined by tests on a model by measuring the sub-atmospheric pressures at these points.

The general arrangement for the necessary measurements is shown diagrammatically in Fig. 5 and are also to be seen in the photograph in Fig. 6. The pressure before the reduction was measured by a mercury piezometer, the velocity pressure at this point being disregarded; at an opening of 100% it amounts to approximately 0·04 *H*. This deviation from reality was partly eliminated by the fact that the loss of pressure in the reduction piece has not been considered. Air is admitted through hollow ribs into the space inside the jet and at the point of contact between the moving piston and fixed core of the valve.

The force acting upon the closing piston was determined by balancing it for certain degrees of opening with various locations of the equalising holes in the piston. Measurements were made altogether at four

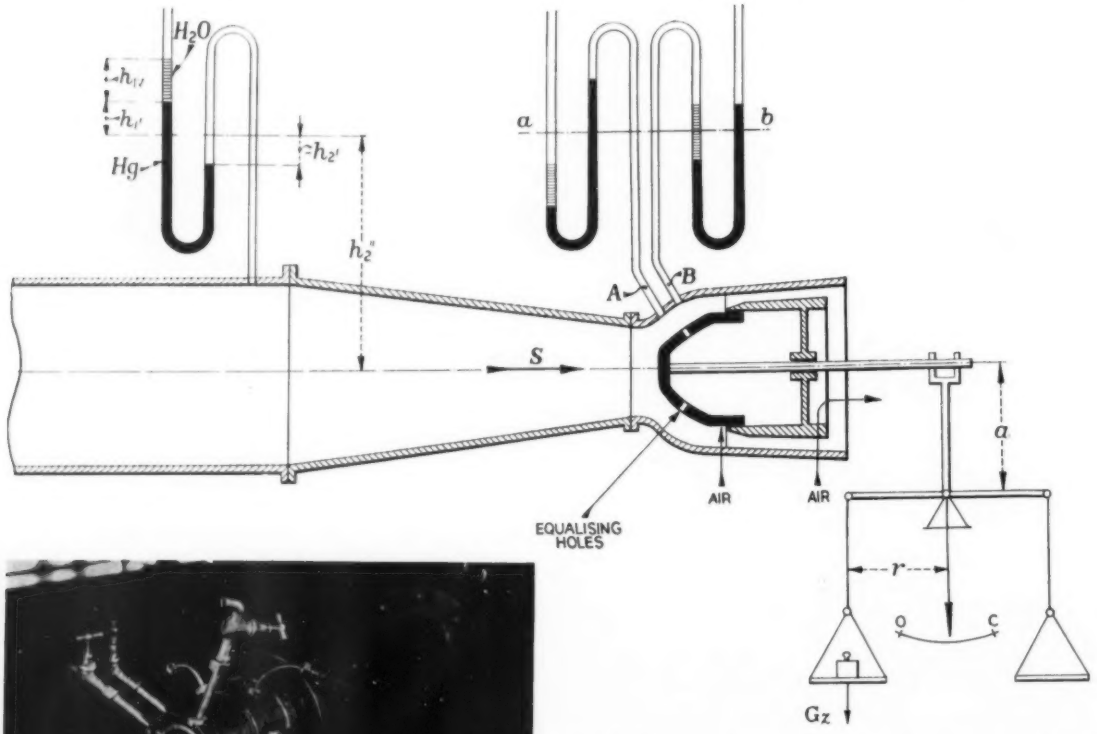


Fig. 5. Pressure from measured values: $H = h_1' + h_2' + h_{1v} + h_2''$, *entered in metres of water column. Force on piston for* $H = 10$ *metres;* $S = \frac{10}{H} \frac{r}{a} Gz$.

Fig. 6

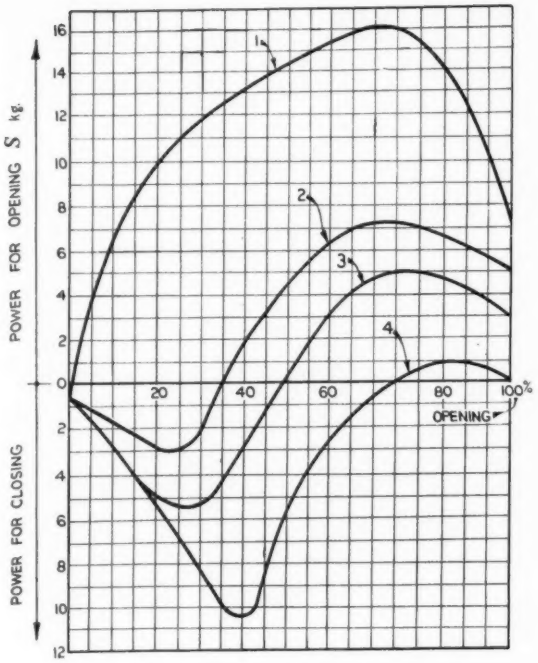


Fig. 7

different positions of equalising holes of 3 mm. diameter. There were always two holes radially opposite each other at the same radius. The results of the measurements are recorded in the graph in Fig. 7, which shows the force acting upon the piston plotted against the opening of the valve at a constant head $H = 10$ m. The shape of the curves proves how great is the influence of the position of equalising holes on the magnitude of the force acting upon the piston and its variation with the degree of opening. When the equalising holes are arranged at a greater distance from the centre (curve No. 1) the valve is under an opening force during practically its entire stroke. The maximum force upon the piston acts at 75% opening. When the holes are arranged near the centre (curve No. 4) the piston is under a closing force during most of its stroke and the force is at its maximum at approximately 40% of the stroke. With the holes arranged on a diameter corresponding to curve No. 3, the force is at a minimum and it is approximately equal for opening and for closing. As has been verified by measurement, the force on the piston is directly proportional to the head, so that the force on the model S_2 can be converted to the actual force S_1 in accordance with the relation:—

$S_1 = \lambda^2 \frac{H_1}{H_2} S_2$ where $\lambda = \frac{D_1}{D_2}$ = scale of geometrical similarity.

The index 1 in the equation refers to the prototype and the index 2 to the model.

The pressures at points *A*, *B* shown in Fig. 5 were measured by mercury piezometers at an approximately constant head. The pressures at these points are plotted against the opening of the valve at a head $H = 10$ m. in the graph in Fig. 8. At point *A* negative pressures were found in the range of openings of 65 to 100%. At lower openings there were overpressures. The negative pressure reaches its maximum depression at an opening of 75%. At point *B* there was a negative pressure in the range of openings from 66 to 63%, with a minimum at 69% opening.

The negative pressures on the model can be converted to the prototype values on the assumption that they are proportional to the head, as deduced below (Fig. 9).

The static pressure at point *A*, when the water is flowing, is lower than the static pressure at that point

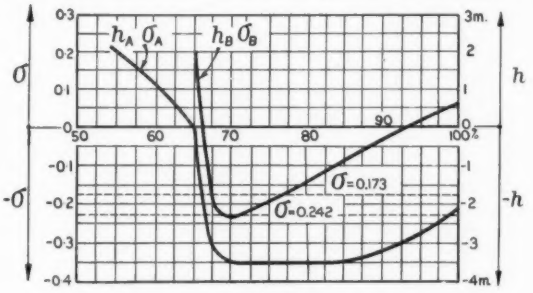


Fig. 8

when the valve is closed, by velocity pressure $\frac{c^2}{2g}$:—

$h = h_1 - x - \frac{c^2}{2g}$ where, when $(H + x')$ is substituted for $(h_1 - x)$, then $h = H \div x' - \frac{c^2}{2g}$. The values of h, h_1, x, x', H are evident from Fig. 9.

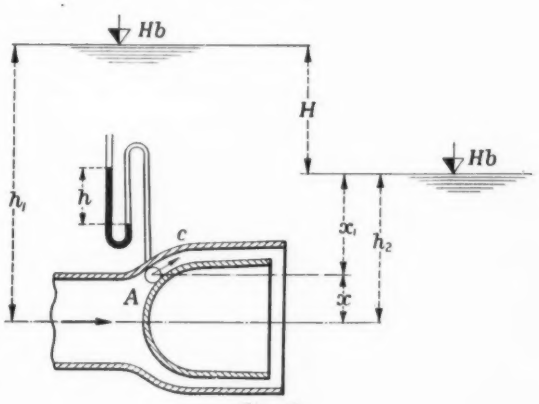


Fig. 9

The velocity at point *A* can be expressed as follows: $c = \sqrt{2gH}$. When this is substituted into the previous equation

$$h = H \div x' - \alpha^2 H = (1 - \alpha^2) H + x'.$$

The sub-atmospheric pressures were measured at a free discharge, i.e. $x' = 0$, so that the previous expression for h is simplified to the expression $h = (1 - \alpha^2) H$.

From this equation $\frac{H}{h} = 1 - \alpha^2 = \text{const.} = \sigma$ or $h = \sigma H$, i.e. the pressure at point *A* is directly proportional to the head.

In the previous equations the barometric pressure has not been considered, so that h must be taken in

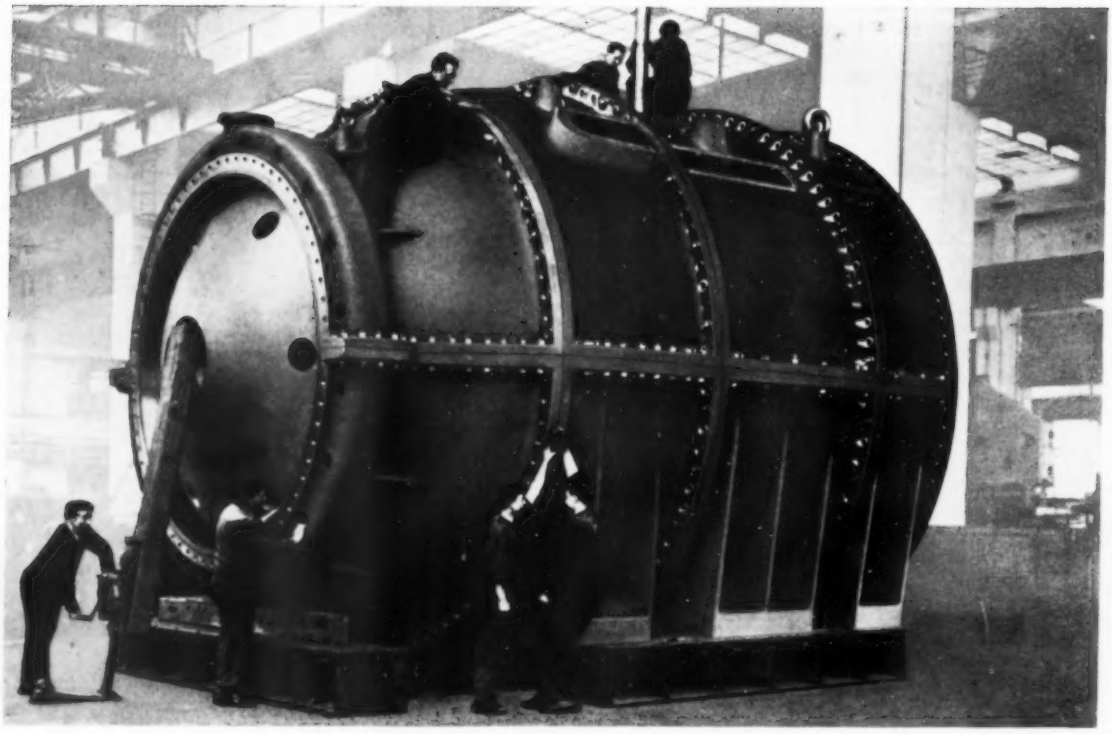


Fig. 10

relation to the barometric pressure. If, in the prototype, the tailwater level varies so that, at maximum tailwater, the discharge is under water, the negative pressure calculated from the model is $h = \sigma H + x'$, i.e. the depression below atmosphere is decreased by the column of water above the discharge jet.

It is therefore possible to find out, from the negative pressure measured at point A, up to how high a head this valve can be used without risk of cavitation, for, if cavitation is to be avoided, the sub-atmospheric pressure must not be lower than the corresponding pressure of saturated water vapour at the corresponding temperature. Assuming a pressure of saturated water vapour of approximately 0·3 m. of water column, which corresponds to a temperature of 24°C., the permissible absolute pressure from the cavitation standpoint must not be less than − 9·7 m. Thus, at a measured $\sigma A = 0{\cdot}354$ the maximum permissible head

$$H_{\max.} = -\frac{h_A}{\sigma_A} = \frac{-9{\cdot}7}{-0{\cdot}354} = 27{\cdot}4 \text{ m.}$$

At a higher head cavitation would take place at point A and it is therefore necessary to admit air. This partly reduces the capacity of the outlet, but smooth operation of the valve is ensured. According to measurements carried out on a 60 mm. diameter model, this reduction of capacity at $H = 10$ m. is only about 1%.

In order to verify the results of the tests on the model, let us quote the practical example of two guided Johnson valves of 1,600 mm. diameter. The valves discharge water under a head of 40 m., which corresponds, at the beginning of cavitation, to:—

$$\sigma = \frac{h}{H} = \frac{-9{\cdot}7}{40} = -0{\cdot}242.$$

It will be seen from Fig. 8 that at point A cavitation would occur within a range of opening of 67 to 99%. In reality, cavitation did manifest itself by vibration and knocking which took place from an opening of 75% to almost 100%, and the discharge was again smooth with the valve fully open.

Fig. 11 shows the model valve in operation at full opening under a head of 10 m., and Fig. 12 at an opening of about 20% under a head of 20 m. Here the influence of the ribs on the shape of the jet is apparent, the jet being divided into eight branches.

To ensure the stability of the flow when discharging below tailwater level, further tests were made on a smaller model, of 40 mm. diameter, geometrically similar to the 60 mm. model. The controlling force and negative pressures were measured. This model was cut length-wise, ground flat along its axis and attached in a water-tight fit to a plate of synthetic glass, through which the flow could be observed. The model was enclosed in a welded box fitted with a pressure gauge, a deaerating cock and an outlet provided with a throttle valve by means of which counterpressures could be arranged to correspond to the position of the valve below the tailwater level. The admission of air into the water jet was arranged in the same way as on the larger model, but in addition air was also admitted at the points at which negative pressures were measured on the larger model.

During discharge at a level which had been lowered to the centreline of the valve by opening the outlet valve, the jet was smooth and had an ample

supply of air. When the box was flooded, however, considerable eddying occurred behind the discharge section inside the jet and reached the inside of the valve. With growing counterpressures in the box, this eddying decreased until, at a counterpressure 10 m. they disappeared almost completely, and, at the points of admission of air into the discharge and of negative pressure on the inlet part, an overpressure occurred.

With the box flooded and zero counterpressure, and maximum eddying behind the discharge section, dyes were injected so that they could be better observed. The dyeing was achieved by drawing a dyed solution to this point instead of air. At the points where it remained still the solution produced colouring of the synthetic glass plate and thus marked on the inside of this plate the contour of the space occupied by the whirl. An extension was then formed with a cross section in accordance with this contour as a continuation of the fixed inner part of the core of the valve in the direction of the outlet.

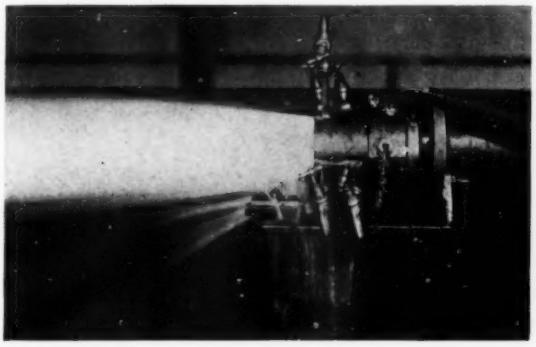


Fig. 11

A supply of air was then arranged around the operating spindle by means of a fairly large semicircular recess in the extension. This improved the flow and the whirls almost completely disappeared.

This arrangement of the extension, which would be ideal from the design point of view, could only be realised with difficulty in practice in view of the large dimensions of the valve. Therefore a shorter extension was manufactured of about half the length. Whirls again appeared behind the valve, but by no means to the same extent as without any extension, so that this solution proved very satisfactory for the actual valve.

After these and further experiments had been carried out to verify the suitability of the shape of the closing body, etc., the shape of this extension was retained as being the most suitable in design for a Johnson valve constructed for a dam built in Czechoslovakia. This dam, impounding a head of 56 m., will be fitted with two valves 3,000 mm. in diameter and for a flow of 200 cu. m. per sec. at full head. During the highest tailwater levels experienced, the centre-line of the discharge jet is approximately 7 m. below water level. The axial force against the piston is obtained by conversion from the model:—

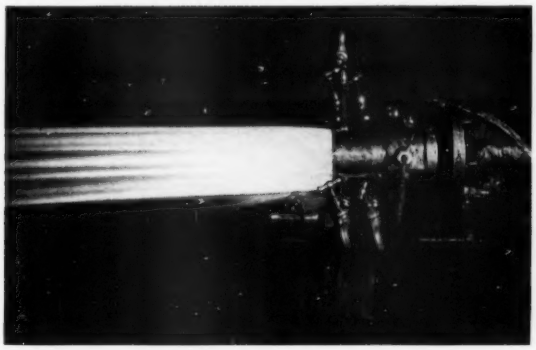


Fig. 12

$$S_1 = \left(\frac{D_1}{D_2}\right)^2 \frac{H_1}{H_2} S_2 = \left(\frac{3000}{60}\right)^2 \frac{56}{10} \times 5{\cdot}5 = 77000 \text{ kg.}$$

An electric motor of 20 kW rating closes the valve from its fully open position in 16 minutes. Apart from air admission into the hollow space of the jet and at the points of contact between the fixed and moving parts, air also had to be abundantly admitted into the inlet at those points at which measurement on the model indicated negative pressures to exist, for, according to Fig. 8, there appears to be $\sigma = \frac{-9{\cdot}7-}{56} =$ 0·173 with a risk of cavitation within a range of openings of 66·5 to 100%.

Fig. 10 shows the assembly work of this valve in the shops.

Ferranti Progress

Ferranti Limited report that they supplied a total of nearly 2¾ million kVA of large power transformers during last year. Of those shipped abroad there were two 189,000 kVA, three-phase banks of generator transformers for the Dalles Dam project, U.S.A., one for 230 kV operation and one for 115 kV operation; five 60 MVA 132 kV, three-phase transformers for the Electricity Commission of New South Wales; one 80,000 kVA, 154 kV, three-phase bank of transformers and spare unit for the Tennessee Valley Authority, U.S.A., and several transformers of up to 36 MVA rating for South Africa and India. An outstanding order received during the year covers the supply of 13 transformers for the Power Authority of the State of New York, Niagara power project, and Lewiston power plant. This contract, comprising seven 180,000 kVA, 240 kV, three-phase transformers and six 180,000 kVA, 120 kV three-phase transformers, is believed to be the largest overseas transformer order ever to be placed in the United Kingdom, surpassing the contract for the transmission transformers for the Kariba hydro-electric scheme, Southern Rhodesia.

Ferranti Limited are pursuing a vigorous programme of research and development, particularly into the problems of noise, the use of silicone materials, cold-rolled steels, general development work on distribution transformers, high-voltage and impulse testing plants, and moving-coil voltage regulators for transmission and distribution systems at home and abroad.

The Correlation of Nuclear, Thermal and Pumped-Storage Capacity

Problems requiring knowledge of the shape of the duration curve and the combination of nuclear base load with the corresponding pumped-storage capacity are discussed in this section

By DR. CHARLES JAEGER

PART TWO

IT is proposed to replace the system load curves to be studied by an equivalent load-duration curve. The actual curve is then replaced by an equivalent mathematical curve of a type which is suitable for analysis. The conditions are that the actual load-duration curve and its mathematical equivalent must show the same load factor θ and the same ratio θ_0 = minimum/maximum load or $\theta_0 = m/M$ (N = load at the time t, M = maximum load, m = minimum load).

Choice of Mathematical Formula

Two mathematical formulae can be recommended for further analysis:

(*a*) The well-known formula based on the probability theory:

$$N = M\left[\theta_0 + \frac{2}{\sqrt{\pi}}(\theta-\theta_0)\left(\log_e\frac{T}{t}\right)^{1/2}\right]$$

where T is the period considered and t/T the relative time, or (*b*) the formula of Soschinski

$$N = M[1-(1-\theta_0)t^{(\theta-\theta_0)/(1-\theta)}] \quad \ldots(3)^*$$

where t is the relative time ($T = 1$). See Fig. 14.

The relationship $\theta_0 = \theta^{2\cdot1}$ to about $\theta_0 = \theta^{2\cdot5}$ is recommended if θ_0 is not known from existing statistics.

The formula of Soschinski has been found to agree closely with the shape of actual load-duration curves, for a given value θ, provided the correct value θ_0 is known. This formula has, therefore, been adopted for further enquiries, using the approximate value $\theta_0 = \theta^2$ for numerical computations.

The basic problem to be solved first is that of adjusting the power generation for a given system load curve, defined by the two values θ and θ_0, when only two means of production are considered. For this problem it is assumed that the base load N_0 extends over the period T, the load variation over the same period being entirely covered by the pumped-storage scheme.

Such conditions would be ideal for a grid entirely fed by nuclear power, and pumped storage designed to match it.

The Soschinski formula immediately yields:

$$t = \left[\frac{M-N}{M(1-\theta_0)}\right]^{1-\theta/\theta-\theta_0} \quad \ldots(4)$$

Let us assume that (Fig. 15).

$N = N_1$ for $t = t_1$

and $M-N_1 = N_1^* = (1-\theta_0)t_1^{\theta-\theta_0/1-\theta}$

then $$t_1 = \left[\frac{N_1^*}{M(1-\theta_0)}\right]^{1-\theta/\theta-\theta_0} \quad \ldots(5)$$

The area A_1^* is equal to

$$A_1^* = Mt_1 - \int_0^{t_1} N\,dt = (1-\theta_0)M\int_0^{t_1} t^{\theta-\theta_0/1-\theta}\,dt$$

$$A_1^* = (1-\theta)Mt_1^{1-\theta_0/1-\theta}$$

Similarly area A_1 is

$$A_1 = N_1^*t_1 - (1-\theta)Mt_1^{1-\theta_0/1-\theta} \quad \ldots(6)$$

Eliminating t_1

$$A_1 = \left[\frac{N_1^*}{M(1-\theta_0)}\right]^{1-\theta/\theta-\theta_0} \times$$

$$\times N_1^* - (1-\theta)M\left[\frac{N_1^*}{M(1-\theta_0)}\right]^{1-\theta_0/\theta-\theta_0}$$

$$= (\theta-\theta_0)M^{-(1-\theta/\theta-\theta_0)}\left(\frac{N_1^*}{1-\theta_0}\right)^{1-\theta_0/\theta-\theta_0} \quad \ldots(7)$$

From this equation, we get

$$N_1^* = (1-\theta_0)\left(\frac{A_1}{\theta-\theta_0}\right)^{\theta-\theta_0/1-\theta_0} M^{1-\theta/1-\theta_0} \quad \ldots(8)$$

Similarly the area A_2 on Fig. 16 has to be calculated. The equations (3) and (4) immediately yield.

$$N_2 = M[1-(1-\theta_0)t_2^{\theta-\theta_0/1-\theta}] \quad \ldots(9)$$

and $$t_2 = \left[\frac{M-N_2}{M(1-\theta_0)}\right]^{1-\theta/\theta-\theta_0} \quad \ldots(10)$$

Furthermore

$$A_2 = N_2(1-t_2) - M\int_{t_2}^{T=1} [1-(1-\theta_0)t^{\theta-\theta/\theta 1-\theta}]\,dt$$

$$A_2 = N_2(1-t_2) - M(1-t_2) + M(1-\theta)[1-t_2^{1-\theta_0/1-\theta}] \quad \ldots(11)$$

or $$A_2 = N_2(1-t_2) - M(\theta-t_2) - (1-\theta) - Mt_2^{1-\theta_0/1-\theta} \quad \ldots(11a)$$

If η is the combined efficiency of pumps and turbines then the condition for pure or true pumped storage (Fig. 17) is: $A_1 = \eta A_2 \qquad \eta < 1$

* Throughout this article the terms of the indices $(\theta-t_0)/(1-\theta)$, etc., should be understood to be bracketed as shown in equation (3).—ED. W.P.

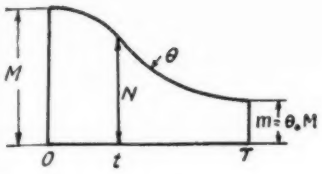


FIG. 14.

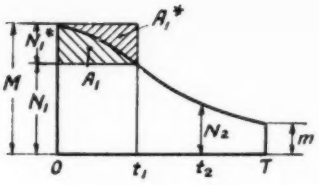


FIG. 15.

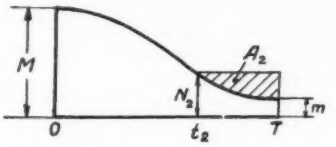


FIG. 16.

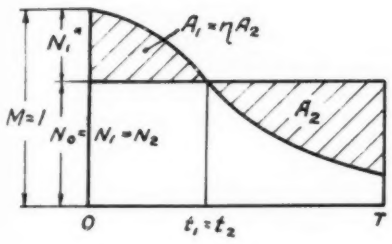


FIG. 17.

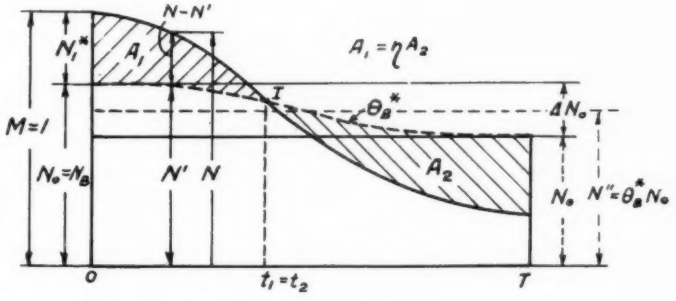
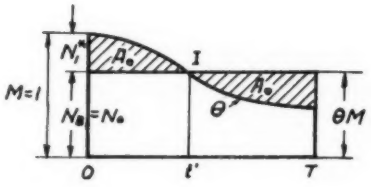
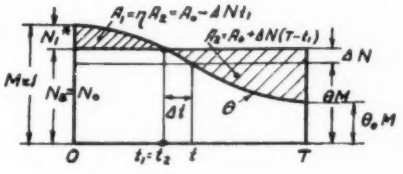


FIG. 18.



a)



b)

FIG. 19.

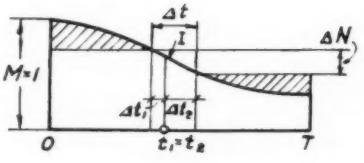


FIG. 20

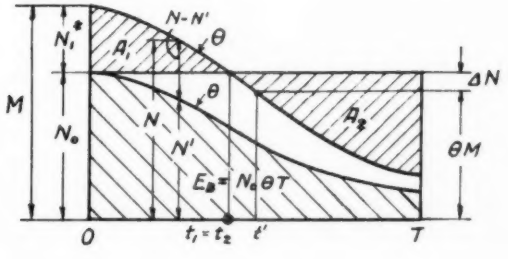


FIG. 21.

The basic problem as stated at the opening of this section requires the conditions:

$$t_1 = t_2$$
$$N_1 = N_2 = N_0 \quad \text{with} \quad A_1 = \eta A_2$$

to be fulfilled. This can be obtained only by trial and error, assuming a certain value t_2 which yields N_2 and A_2, then writing $A_1 = \eta A_2$, which gives N_1^* and N_1 and then t_1. If $t_1 = t_2$, the solution is the correct one, knowing A_1, check that

$$N_1^* = (1-\theta_0)\left(\frac{A_1}{\theta-\theta_0}\right)^{\theta-\theta_0/1-\theta_0} M^{1-\theta/1-\theta_0}$$

and $$t_1 = \left[\frac{N_1^*}{M(1-\theta_0)}\right]^{1-\theta/\theta-\theta_0}$$

The present yearly load factor of the British system varies between 0·386 for London and 0·593 for South Wales.† Daily load factors for summer days are likely to be lower. As the present enquiry is a purely theoretical one and does not refer to any specified system, the calculations have been carried out for $\theta = 0{\cdot}55$, 0·65 and 0·75 with $\theta_0 \cong \theta^2$ as indicated in the

† Central Electricity Authority, Annual Report and Accounts 1956/7, p. 48.

TABLE I. MAIN RESULTS FOR DURATION CURVES CHARACTERISED BY θ AND θ_0

θ	θ_0	$t_1=t_2$	A_1	A_2	N_1^*	θ_1^*	$N_2=N_0$	N_2^*	ΔN	V in m³ for $H=100^m$	V in m³ for $H=300^m$
0·75	0·55	0·445	0·043	0·062	0·235	0·183	0·765	0·215	0·015	448,000	149,000
0·65	0·42	0·42	0·055	0·079	0·33	0·167	0·67	0·265	0·020	550,000	183,000
0·55	0·303	0·40	0·0605	0·0865	0·42	0·128	0·58	0·224	0·030	630,000	260,000

N_1^* is the required turbine capacity and N_2^* the pump capacity.

accompanying Table 1. All the values A_1, A_2, N_1^* (generating capacity of the pumping station) N_2^* (pumping capacity) are in relative values (calculated for $M = 1$). The times are also in relative values ($T = 1$). For the purpose of estimating the required storage volumes a theoretical combined capacity $M = 100$ MW and $T = 24$ hours have been assumed for the system under consideration. This storage capacity is calculated with the formula‡

$$V = \frac{270 t_T}{\eta_T H} N_1^* \frac{1}{0{\cdot}736}$$

(metric system, N_1^* given in kW, V in m^3)

t_T being the time during which the turbines work at full load.

In relative values, this time is $\theta_1^* = A_1/N_1^* T$ ($T = 1$). For estimating volumes $T = 24$ hours. The time θ_1^* is also a load factor. In addition η_T is the turbine efficiency taken at $\eta_T = 0{\cdot}85$.

The results have to satisfy equation (1) (for $M = 1$, $T = 1$)

$$\theta = N_1^* \theta_1^* + N_0 \theta_1 \quad \text{where} \quad \theta_1^* = A_1/N_1^*$$

where $N_0\theta_1 = N_0 - A_2$ or $\theta_1 = 1 - A_2/N_0$

So that

$$\theta = A_1 + N_0 - A_2 = A_2(\eta - 1) + N_0$$
$$N_0 = \theta + A_2(1-\eta)$$

As A_2 is small a good guess for $N_0 = N_1 = N_2$ can be made at a start.

Generalisation of the Method

Case of the base-load factor θ_B being smaller than unity ($\theta < \theta_B < 1$). The required demand load factor for the two combined stations is still θ. The base load is now supposed to vary progressively over the period T from a maximum value N_0 to a minimum value N'_0. The duration curve representing this load variation can more or less be represented by almost any type of mathematical function. For convenience, it is assumed that N' is of the type (Fig. 18).

$$N' = N'' + \frac{\Delta N_0}{2} \cos \frac{\pi t}{T}$$

For $t = 0$, $N' = N_0 = N'' + \frac{\Delta N_0}{2}$

for $t = T$, $N' = N'_0 = N'' - \frac{\Delta N_0}{2}$

Therefore $N'' = \frac{N_0 + N'_0}{2}$

$$\Delta N_0 = N_0 - N'_0$$

and $N' = \frac{N_0 + N'_0}{2} + \frac{N_0 - N'_0}{2} \cos \frac{\pi t}{T}$

‡ Ch. Jaeger. "The Economics of Pumped Storage." WATER POWER, February/March 1957, formula (5).

Further, we define a base-load station load factor

$$\theta_B^* = \frac{\int_0^T N' \, dt}{N_0 T} = \frac{N_0 + N'_0}{2N_0} = \frac{N''}{N_0} = 1 - \frac{\Delta N_0}{2N_0}$$

The area under the curve N' is obviously

$$\int_0^T N' \, dt = \int_0^T N'' \, dt + \frac{\Delta N_0}{2} \int_0^T \cos \frac{\pi t}{T} dt = N''T = \theta_B^* N_0 T$$

which is $(1 - \theta_B^*) N_0 T$ less than for $N_1 = N_0 =$ constant.

The point I, where the two curves N and N' intersect for $t = t_1$ is given by:

$$M[1 - (1-\theta_0) t_1^{\theta - \theta_0/1-\theta}] = N'' + \frac{\Delta N_0}{2} \cos \frac{\pi t_1}{T}$$
$$= \theta_B^* N_0 + \frac{\Delta N_0}{2} \cos \frac{\pi t_1}{T}$$

This transcendental equation is best solved by trial and error or graphically. More generally for any value $t < t_1$

$$N - N' = M[1 - (1-\theta_0) t^{\theta - \theta_0/1-\theta}] - \left(N'' + \frac{\Delta N_0}{2} \cos \frac{\pi t}{T} \right)$$

and

$$A_1 = \int_0^{t_1} (N - N') \, dt$$
$$= M \int_0^{t_1} [1 - (1-\theta_0) t^{\theta - \theta_0/1-\theta}] \, dt -$$
$$- \int_0^{t_1} \left(N'' + \frac{\Delta N_0}{2} \cos \frac{\pi t}{T} \right) dt$$

and for $M = 1$, $T = 1$

$$A_1 = (1 - N'') t_1 - (1-\theta_0) t_1^{1 - \theta_0/1-\theta} - \frac{\Delta N_0}{2\pi} \sin \frac{\pi t}{T}$$
$$= A'_1 - \frac{\Delta N_0}{2\pi} \sin \pi t_1 \qquad \ldots(14)$$

Similarly for $t = t_2$ and $T = 1$

$$A_2 = \int_{t_2}^T (N' - N) \, dt$$
$$= \int_{t_2}^T (N'' + \frac{\Delta N_0}{2\pi} \cos \frac{\pi t}{T} \, dt - \int_{t_2}^T [1 - (1-\theta_0) t^{\theta - \theta_0/1-\theta}] \, dt$$

$$A_2 = (1 - t_2)(N'' - 1) - \frac{\Delta N_0}{2\pi} \sin \pi t_2 +$$
$$+ (1-\theta_0)[1 - t_2^{1 - \theta_0/1-\theta}]$$
$$= A'_2 + \frac{\Delta N_0}{2\pi} \sin \pi t_2 \qquad \ldots(15)$$

The problem is properly solved if for $t_1 = t_2$ (point I),

$$A_1 = \eta A_2$$

or $$A'_1 - \eta A'_2 = (1-\eta)\frac{\Delta N_0}{2\pi}\sin \pi t_1$$

In practical cases θ and $\theta_B{}^*$ are given load factors. A base-load capacity $N_0 = N_B$ is estimated, which then yields $N'' = \theta_B{}^* N_0$ and $\Delta N_0/2 = N_0 - N''$. A trial and error calculation yields the point of intersection $t_1 = t_2$ of the two duration curves determined by θ and $\theta^*{}_B$. The areas A_1 and A_2 are calculated for this point $t_1 = t_2$ and the value $\eta = A_1/A_2$ checked. If it is not the expected one, a new trial with a corrected value $N_0 = N_B$ will be more successful. At a start a good guess for $N_0 = N_B$ is given by the formula

$$\theta = \frac{N_1{}^*}{M}\theta_p + \frac{N_B}{M}\theta_B$$

Where $$\theta_p = \frac{A_1}{N_1{}^* T}$$

and $$\theta_B = \frac{N_B \theta_B{}^* T - A_2}{N_B T} = \theta_B{}^* - \frac{A_2}{N_B T}$$

Assuming $M = 1$ and $T = 1$

$$\theta = A_1 + N_B \theta_B{}^* - A_2 = \theta_B{}^* N_B - A_2(1-\eta)$$

or $$N_B = \frac{\theta + A_2(1-\eta)}{\theta_B{}^*} \qquad ...(16)$$

As $A_2\,(1-\eta)$ is usually small and easy to guess, a good value can be assumed for N_B at a start.

Example—Starting with a system load factor $\theta_s = \theta = 0{\cdot}75$ and assuming that the base-load station works with a base-load factor $\theta_B = 0{\cdot}82$ and using proper Soschinski duration curves it was found that the problem is properly solved by the values:

$$N_B = N_0 = 0{\cdot}915 \quad A_1 = 0{\cdot}055$$
$$A_2 = 0{\cdot}0079 \qquad \Delta A = 0{\cdot}0024$$

when $\eta = 0{\cdot}7$.

$N_1{}^*$ is now only 0·085 against 0·235 found when $\theta_B{}^* = 1$ was assumed.

These remarks show the paramount importance of a high load factor to be applied to the base-load station if the whole system is to run efficiently.

When the base load is produced by a run-of-river station, it may happen that the base load N' increases when the system load N decreases. This important alternative case will be dealt with later.

Remarks on the Efficiency Factor

For the purpose of these calculations the efficiency factor has been assumed to be $\eta = 0{\cdot}70$. In a modern installation the purely mechanical efficiency factor of the turbines and pumps would actually be slightly higher. At Reisach pumping station,† $\eta = 0715$, including a factor for total tunnel losses (pumping and generating) and a factor $\eta = 0{\cdot}995$ for house supply. Losses due to transport of electrical energy from the base-load generating station to the pumping station and from the pumping station to the consumer must be included and this would lead to higher total losses and a lower value of η.

The bearing of η on the whole calculation is quite important and a short-cut method has to be developed in order to estimate how the results of the calculations would be modified if η varies.

The total energy produced by the combined station, assuming no losses in the pumping process ($\eta = 1$) is

$$A = MT\theta \quad \text{or} \quad A = \theta$$

if $M = 1$, $T = 1$

Because of losses occurring in transforming the energy, $A_1 = \eta A_2$, a loss $A_2 - A_1 = (1-\eta)A_2 = \Delta A$ occurs.

According to Figs. 19a and 19b

$$A_2 = A_0 + \Delta N(T - t_1)$$
$$A_1 = A_0 - \Delta N t_1$$

and $$A_2 - A_1 = \Delta N.T = \Delta A$$

where A_0 is the area $A_1 = A_2 = A_0$ for $\eta = 1$.

This occurs for a time t'. If $\eta < 1$ then:

$$N_0 = \theta + \Delta N$$

or $$\Delta N = N_0 - \theta = 1 - N_1{}^* - \theta$$

If ΔA is the energy loss caused by $\eta < 1$, ΔN is the corresponding loss of capacity, extending over the whole period T. Therefore, $\Delta N = \Delta A/T = (1-\eta)\ A_2/T$.

According to Fig. 19b

$$A_2 = A_0 + \Delta N\left(T - t' + \frac{\Delta t}{2}\right) \cong A_0 + (T - t')\Delta N$$

$$\Delta N = (1-\eta)\frac{A_2}{T} = \frac{1-\eta}{T}A_0 + \frac{1-\eta}{T}(T-t')\Delta N$$

or $$\Delta N = \frac{(1-\eta)A_0}{[\eta(1-t'/T) + t'/T]}$$

As $\frac{\Delta N}{\Delta t} \cong \left(\frac{dN}{dt}\right)_{t'}$

$$\Delta t = \frac{\Delta N}{(dN/dt)_{t'}}$$

where $N = 1 - (1-\theta_0)t^{9-\theta_0/1-\theta}$

for $M = 1$ and $t = t'$

$$\frac{dN}{dt} = -(\theta - \theta_0)^{1-\theta_0/1-\theta} t^{(\theta-\theta_0/1-\theta)-1}$$

for $M = 1$ and $t = t'$ also.

The problem requires, therefore, the knowledge of A_0 and t', obtainable by direct calculation. This is understandable, as the whole investigation supposes that the shape of the curve N is known.

In order to avoid this lengthy calculation, let us write for two different values η_1, and η_2 of η:

$$\Delta N_1 = \frac{1-\eta_1}{T}A_{21} \quad \text{and} \quad \Delta N_2 = \frac{1-\eta_2}{T}A_{22}$$

In first approximation

$$A_{21} = A_{22} = A_2$$

and $$\Delta N_2 - \Delta N_1 \cong \frac{\eta_1 - \eta_2}{T}A_2$$

or $$\Delta N_2 = \Delta N_1 + \frac{\eta_1 - \eta_2}{T}A_2 \qquad ...(17)$$

With this new value of ΔN_2, a better guess can be made for A_2 and a new estimate of ΔN_2 started if η varies from η_1 to η_2.

Let us assume that for $\theta = 0{\cdot}65$, $\theta_0 = 0{\cdot}42$, $A_2 = 0{\cdot}0785$ and $N_1{}^* = 0{\cdot}33$, $\Delta N_1 = 0{\cdot}020$ for $\eta_1 = 0{\cdot}7$.

For $\eta_2 = 0{\cdot}6$ the value of ΔN_2 is

$$\Delta N_2 = 0{\cdot}020 + \frac{0{\cdot}7 - 0{\cdot}6}{T} \times 0{\cdot}0785$$
$$= 0{\cdot}02 + 0{\cdot}0079 = 0{\cdot}028$$

and the additional loss of energy over a period $T = 1$ would be $\Delta\Delta A = (\Delta N_2 - \Delta N_1)T = 0{\cdot}0079 \cong 0{\cdot}008$ (8 per mil) of the total energy $MT = 1 \times 1 = 1$ or $0{\cdot}008/0{\cdot}65 = 0{\cdot}0122$ (1·22% on the energy really generated which is $MT\theta = 0{\cdot}65$.

The new values are now:

† Energie Versorgung Ostbayern A.G. Das Pumpspeicherwerk Reisach-Rabenleite, Regensburg 1957 p. 59.

$$N_B = \theta + \Delta N = 0{\cdot}65 + 0{\cdot}028 = 0{\cdot}678$$
$$N_1^* = 1 - N_B = 1 - 0{\cdot}678 = 0{\cdot}322$$
$$A_2 = 0{\cdot}0785 + 0{\cdot}0079 = 0{\cdot}0864$$
$$A_1 = 0{\cdot}6 \times 0{\cdot}0864 = 0{\cdot}05214$$

If transmission losses over a distance have to be considered, Table II published by B. Jansen and K. H. Haagen can be used.

TABLE II. TABLE OF ENERGY LOSSES IN % FOR FULLY LOADED TRANSMISSION LINES, FOR 100 KM. (62 MILES)

Line	6 Cables	6 Cables	6 Bunches of Cables
Voltage (kV) ..	110	220	380
Load (MW) ..	90	250	1200
Line losses ..	5·9	2·2	1·25
Corona losses ..	—	0·4	0·1
Total line losses ..	5·9	2·6	1·35
End losses ..	—	1·0	1·0

Effect of a Time Lag in putting the Peak-Load Station into Operation (Fig. 20)

It is never possible to put any station into operation instantaneously. There is always a time lag Δt between the two operations and usually $\Delta t = \Delta t_1 + \Delta t_2$ where Δt_1 refers to an operation on the turbines and Δt_2 on the pumps (or *vice versa*).

In Fig. 20 it is assumed that this time lag Δt occurs only once in the period T and it has been divided on both sides of the time $t_1 = t_2$. According to what has been established in the previous paragraph this time lag causes a loss of capacity ΔN equal to:

$$\Delta N = \left(\frac{dN}{dt}\right)_{t_1} \Delta t$$

or an additional total loss $\Delta E = \Delta N T$ over the period T. In order to make good at least part of the loss, the capacity of the turbine and the volume of the storage could be increased.

The time lags for those pumping stations, combined with those incidental to generation, are extremely short as compared with the time lags required by peak-load steam stations, and this is a considerable advantage in favour of pumped-storage capacity.

Table III gives the times according to the Energie Versorgung Ostbayern A.G.

TABLE III
APPROXIMATE TIME LAG FOR PUMPED-STORAGE SYSTEM

Putting a turbine into operation from standstill to full load	130 sec.
Putting pump into operation from standstill to full load	180 sec.
Passage from full turbine generation to full pumping capacity	70 sec.
Passage from full pumping capacity to full generating capacity	60 sec.

No steam peak-load station can compete with such figures.

Combination of Base Load plus Peak Load and System Efficiency (Fig. 21)

A base-load station is supposed to have a capacity N_0. The load factor is θ and the total energy delivered by this base-load station $E_B = N_0 \theta T$, T being a day, a week or a year.

Let us assume that a suitable peak pumping station is directly connected to the base-load station so that the base-load station now works with a load factor equal to one. If η is the efficiency factor for transformation of energy the capacity of the peak-load station will be N_1^*, calculated as before and the total installed capacity will be

$$M = N_0 + N_1^*$$

If the combined stations work with the same load factor θ as the base-load station was working with before, the condition

$$\theta = \frac{\int_0^T N\,dt}{MT} = \frac{\int_0^T N'\,dt}{N_0 T}$$

will be satisfied if

$$\frac{N}{M} = \frac{N'}{N_0}$$

If we, therefore, trace the two rectangles

$$\theta MT = \int_0^T N\,dt \quad \text{and} \quad \theta N_0 T = \int_0^T N'\,dt$$

the horizontal sides of the rectangle will cut the curves N and N' for a same value t'. Because $A_1 = \eta A_2$ and $\eta < 1$ the horizontal through N_0 cuts the curve N for a time $t_1 = t_2 < t'$ and N_0 will be $N_0 > \theta M$. In fact, as seen before

$$N_0 = \theta M + \Delta N$$

The total energy output of the two combined stations is obviously

$$E_{\text{tot}} = M\theta T = \frac{M}{N_0} E_B$$

The gain in energy is

$$\Delta E = \theta T(M - N_0) = \theta T N_1^*$$

in absolute values and

$$\frac{\Delta E}{E_B} = \frac{\theta T N_1^*}{\theta T N_0} = \frac{N_1^*}{N_0}$$

in relative figures.

Let us consider the case of a load factor $\theta = 0{\cdot}70$ with $M = 100$ MW and $T = 8670$ hours. The base-load station alone would produce

$$E_B = 100{,}000 \times 0{\cdot}70 \times 8760$$
$$= 613{,}000{,}000 \text{ kWh a year}$$

Interpolation from Table I gives $N_1^* = 0{\cdot}275$, $A_1 = 0{\cdot}049$, $A_2 = 0{\cdot}07$.

Adding a pumping station with a capacity of $N_1^* = 100 \times 0{\cdot}275/0725 = 37{,}900$ kW to total 137,900 kW, will raise the total energy produced to

$$T_{\text{tot}} = 613{,}000{,}000\left(\frac{0{\cdot}275}{0{\cdot}725} + 1\right) = 845{,}000{,}000 \text{ kWh}$$

the increase in energy being 232,000,000 kWh.

The amount of energy used for pumping is

$$E_p = A_2 \times MT = 0{\cdot}07 \times 137{,}900 \times 8760$$
$$= 84{,}600{,}000 \text{ kWh only}$$

Of this $T_{\text{turb}} = \eta A_2 MT = 59{,}300{,}000$ kWh reappears as energy generated in the pumping station. The remainder, $232{,}000{,}000 - 59{,}300{,}000 = 172{,}700{,}000$ kWh arises directly from the better utilisation of the base-load station.

If the cost of producing E_B kWh a year in a base-load station with N_B installed capacity is

$$K = mN_B + nE_B \qquad \ldots(19)$$

or K/E_B per kWh.

The cost for producing E_{tot} kWh in the two stations is now

$$K_1 = mN_B + m''N_1^* + n'N_B T + n''E_{\text{turb}} \qquad \ldots(20)$$

or K_1/E_{tot} per kWh.

In these formulae m, m'', n', n'' are conversion factors. The real increase in system efficiency will be figured by comparing K_1/E_{tot} and K/E_B. Price estimates of this type will now be attempted.

(To be continued)

Power Penstocks

Analysis of various penstock designs and construction methods
by P. J. Bier, Consulting Engineer, formerly in charge of
penstock design of the U.S. Bureau of Reclamation

PART TWO

Design "B"

In this design Dresser-coupled field girth joints and 120° saddle supports are provided and the shell thicknesses are proportioned for the internal pressure.

Description. This penstock design is shown in Fig. 7. It consists of pipe sections shop fabricated in 40 ft. erection lengths for installation in the field, all with Dresser coupled field girth joints of the type as shown in Fig. 8. The upper half of the penstock is fabricated from a carbon steel of A285 gr. C, and the lower half from a carbon-silicon steel of A212 gr. B. All welds in the pipe shell are radiographed, but no stress relieving is used. This permits a joint efficiency of 90% throughout the penstock and a net design stress of $13,750 \times .90 = 12,375$ lb. per sq. in. for the A285 steel and a stress of $17,500 \times .90 = 15,750$ lb. per sq. in. for the A212 steel. With the exception of the upstream,

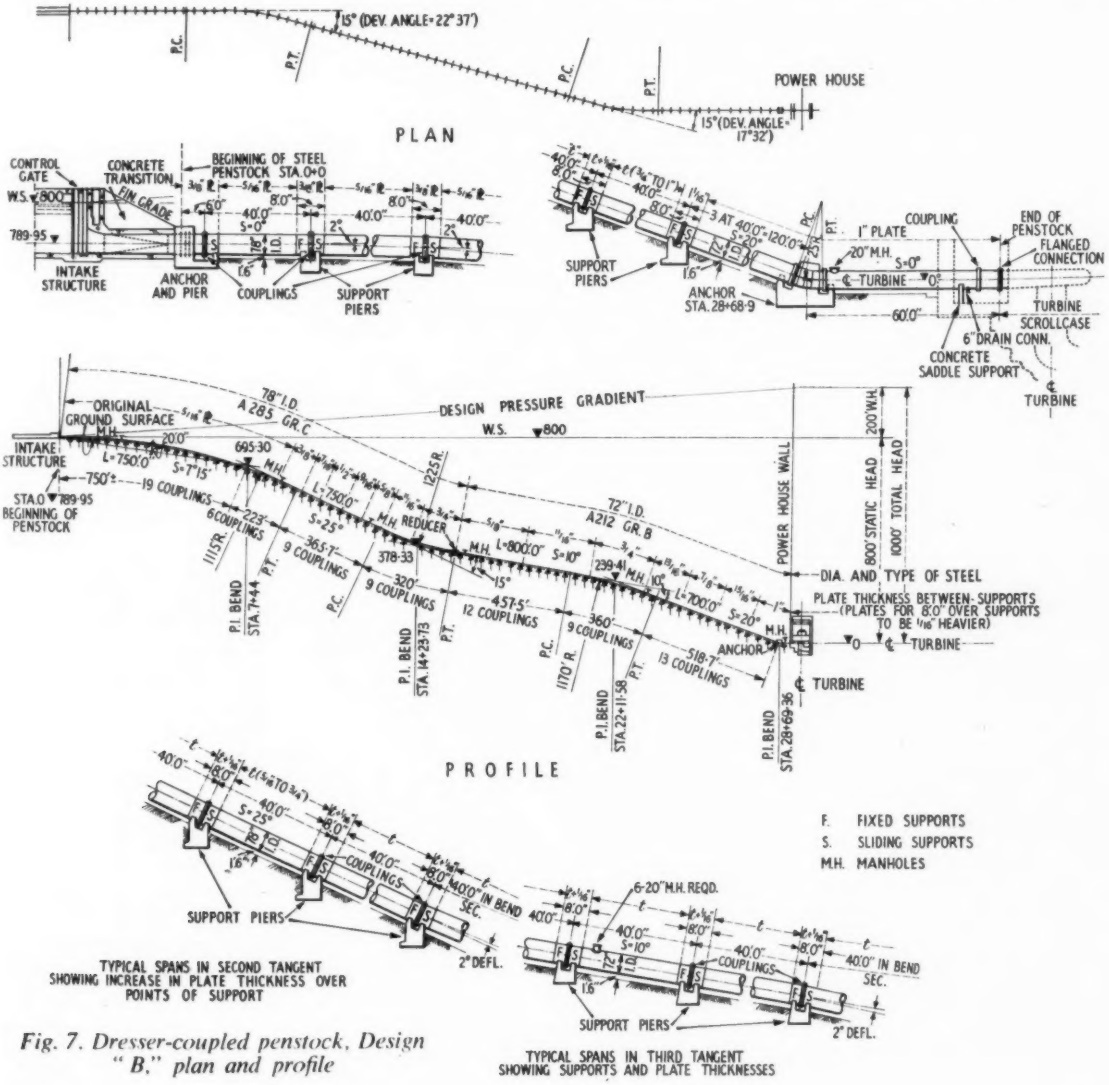


Fig. 7. Dresser-coupled penstock, Design "B," plan and profile

low head end section of the penstock, the pipe shell thickness is proportioned solely for the internal pressure, that is, the static and water-hammer heads as plotted on the profile in Fig. 7. The shell thickness for the upstream-end section was assumed at $\frac{5}{16}$ in. similarly to Design "A," to satisfy the requirements of rigidity. This thickness is also sufficient to resist the circumferential bending moment of M=CPR at the 120° saddle supports as obtained experimentally in tests made at the University of Illinois Experiment Station.[3] Fig. 9 shows a diagram of the variation of circumferential bending moment for the various saddle supports. According to Arriaga's analysis[4] the stresses due to internal pressure and circumferential bending at the saddle supports are not additive, as the latter stress is reduced very appreciably with an increase in internal pressure. Fig. 10 shows the minimum shell thickness required for unstiffened pipe on 90 and 120° saddle supports, based on a stress equal to $\frac{2}{3}$ of the yield point of the steel used. This would require a shell thickness of $\frac{1}{4}$ in. for the 78 in. diameter penstock on a 120° saddle. With a stress of half the yield point of the A285 gr. C. steel, a shell thickness of about $\frac{5}{16}$ in. is required, which is used at the upper end of the penstock. As this circumferential bending stress is merely a temporary stress acting on the pipe shell during filling and draining operations, when there is no internal pressure in the pipe, a high maximum stress is permissible for such a short period.

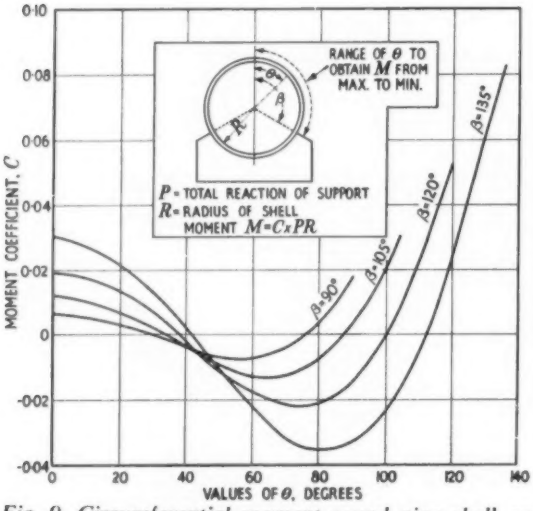


Fig. 9. Circumferential moment round pipe shell on saddle support

No fabricated bends or concrete anchors are used at points of deflection except at the downstream end near the power house. At all other points the changes in direction are effected by deflections in the couplings, which were taken at 2° from each coupling.

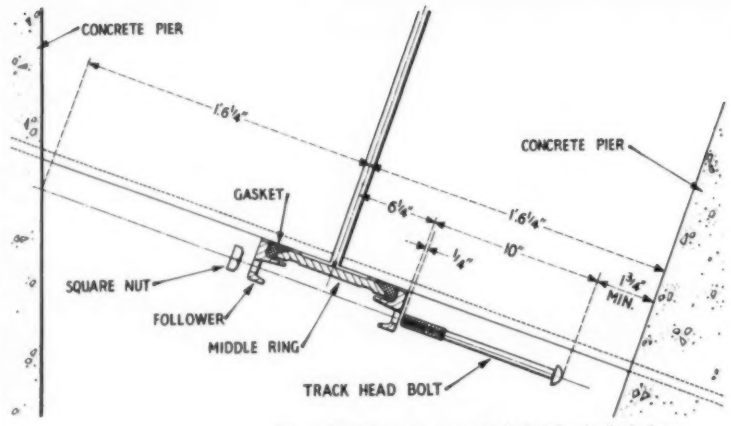


Fig. 8. Dresser-coupled field girth joint

Consequently, the line deflection in degrees divided by 2 determines the number of pipe sections and couplings required to obtain a bend.

Expansion joints. No special expansion joints are required in a Dresser coupled penstock as each coupling functions as an expansion joint for a pipe section. Movements due to temperature changes of 150° F. in a 40 or 50 ft. pipe section are absorbed by the resilience of the rubber gaskets in each coupling. Erection sections of 40 ft. length are used in the design to accommodate the usual fabrication, handling and transportation facilities. Sections of somewhat greater length may in some cases be used if justified by local conditions, provided they do not exceed the capacity of the rubber gaskets to absorb temperature movements without slipping.

Support Piers. Reinforced concrete saddle type piers with 120° contact are used. These saddles are effective in resisting deformations in the pipe shell

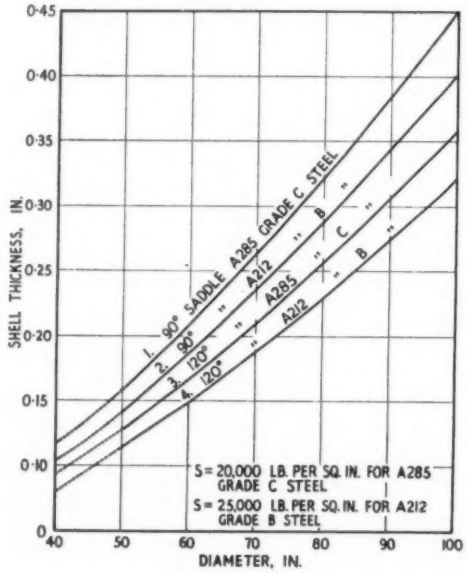

SHELL THICKNESS, IN.
DIAMETER, IN.
1. 90° SADDLE A285 GRADE C STEEL
2. 90° " A212 " B "
3. 120° " A285 " C "
4. 120° " A212 " B "
S = 20,000 LB. PER SQ. IN. FOR A285 GRADE C STEEL
S = 25,000 LB. PER SQ. IN. FOR A212 GRADE B STEEL

Fig. 10. Minimum shell thickness for unstiffened pipe on saddle support

at the points of support which is particularly important during filling and draining when there is no internal pressure. The customary stiffeners and ring girders can be dispensed with as the concrete saddles will function as stiffeners. An increase in shell thickness at the points of support adds to the rigidity of the pipe at the supports. Before installing the penstock on the saddle supports, however, it is necessary to maintain the circularity of the pipe, especially where a minimum shell thickness is used, by temporary braces on the inside.

Reliable data for the analysis of stresses in the pipe shell at saddle supports is very meager and is limited to the experiments made at the University of Illinois. The validity of the results arrived at in these experiments have been questioned by Arriaga, who believes that they are too conservative.

A typical concrete support pier of the saddle type used in this design is shown in Fig. 11. This was designed for the 72 in. penstock on a 20° slope. The pier has two points of support, each forming a 120° saddle for the pipe section supported on it. A Style 38 Dresser coupling connects the adjoining 40 ft. sections in a recess between the two support points of the pier. Each pipe section is anchored at one end to the pier, while the other end is free to slide on the saddle to accommodate the expansion and contraction of the section due to temperature changes. The resistance to sliding (or the sliding coefficient) will determine the temperature force acting on the pier. All saddles must be constructed to a radius to fit the outside surface of the penstock. The tendency to form a sharp kink near the top of the saddle may be reduced or eliminated by rounding out the top edge or tapering it as shown in Fig. 12. All support piers must be designed to carry the pipe and water load of the span to the underlying soil at a safe bearing pressure and to resist the longitudinal frictional forces of the filled pipe section due to temperature changes. Tests performed on steel pipe supported on concrete piers[3] yielded, on average, a frictional coefficient of 0·60 between steel and concrete. It is customary, however, to assume that the resistance to movement varies in the saddle supports due to differences in alignment or surface conditions. The coefficient may be as low as 0·3 on one side of a pier and as high as 0·6 on the other side, resulting in a differential of 0·3, which has been used in determining the longitudinal temperature force acting on the pier. In addition to the temperature force there is the gravity force incidental to the weight of the empty pipe and this has to be added to or subtracted from the temperature force, depending on the direction of the latter force.

The forces in Table I are plotted on the pier shown in Fig. 11. These forces, acting in combination with the concrete and earth load, will give a resultant force intersecting the pier base in its middle third to resist overturn. The 168,000 lb. of total dead load will result in a unit bearing pressure of only 2,500 lb. per sq. ft. on the soil foundation.

TABLE I

Forces acting in the upstream direction	
Temperature force of 20 ft. section of filled pipe (P & W) weighing 51,000 lb. = $51{,}000 \times 0{\cdot}3 =$	15,300 lb.
Gravity force of dead weight of pipe (15,500 lb.) = $p.\sin 20° =$	5,300 lb.
Net upstream force =	10,000 lb.
Forces acting in the downstream direction	
Temperature force of 20 ft. section of filled pipe = $51{,}000 \times 0{\cdot}3 =$	15,300 lb.
Gravity force of dead weight of pipe = $15{,}500 \times 0{\cdot}3420 =$	5,300 lb.
Net downstream force =	20,600 lb.

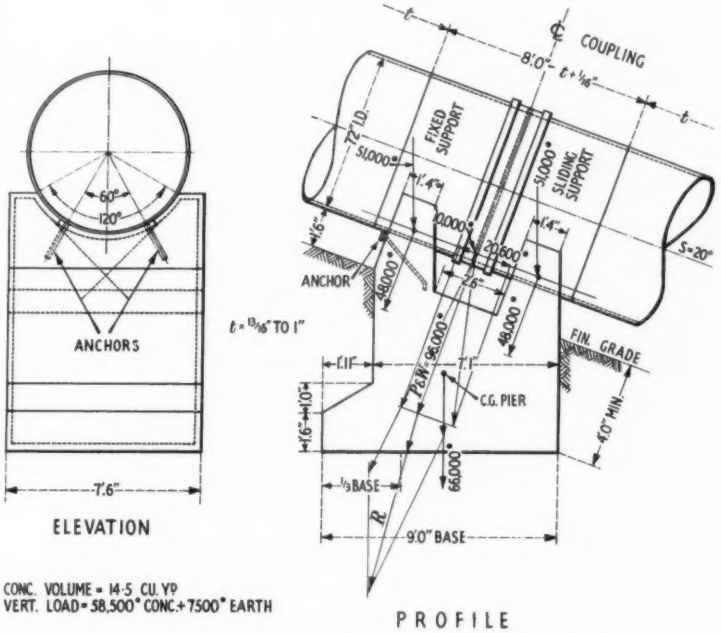


Fig. 11. Typical support pier

With the fabricated bends and anchors eliminated at most pipe deflections, a special type of support pier has been developed for deflections in two directions (combined bends) to resist the forces acting at such points. These forces consist principally of the hydrostatic forces of wAH and the dynamic forces of $\frac{QwV}{g}$, where

w = unit weight of water 62·4 lb. per cu. ft.
A = cross sectional area of the pipe (in sq. ft.)
H = max. head including water hammer (in ft.)
V = velocity of flow in ft. per sec.
g = acceleration due to gravity (32·16)
Q = flow in cusecs.

Fig. 12 shows a typical pier design for the lower combined bend in the 72. in. penstock. A longitudinal force of 1,282,000 lb. is acting from both sides of the

Fig. 12. Support pier for combined head

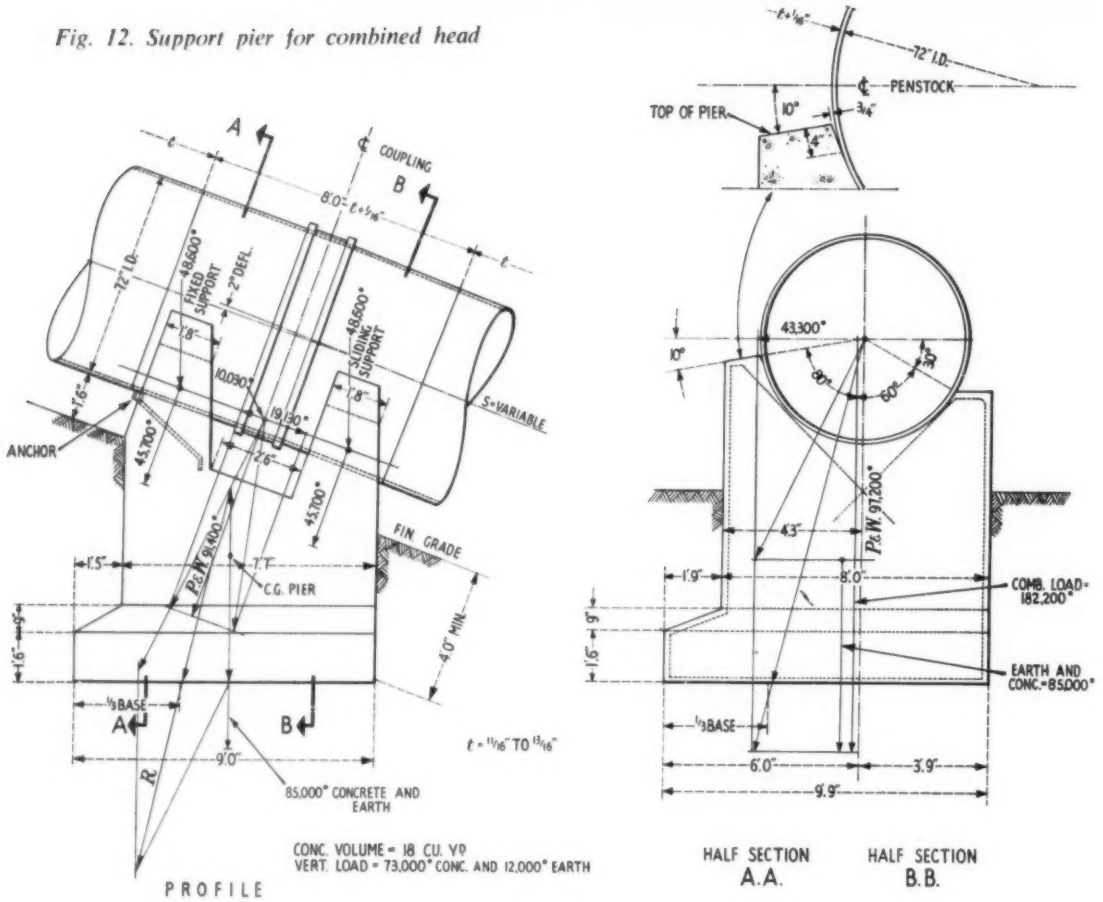
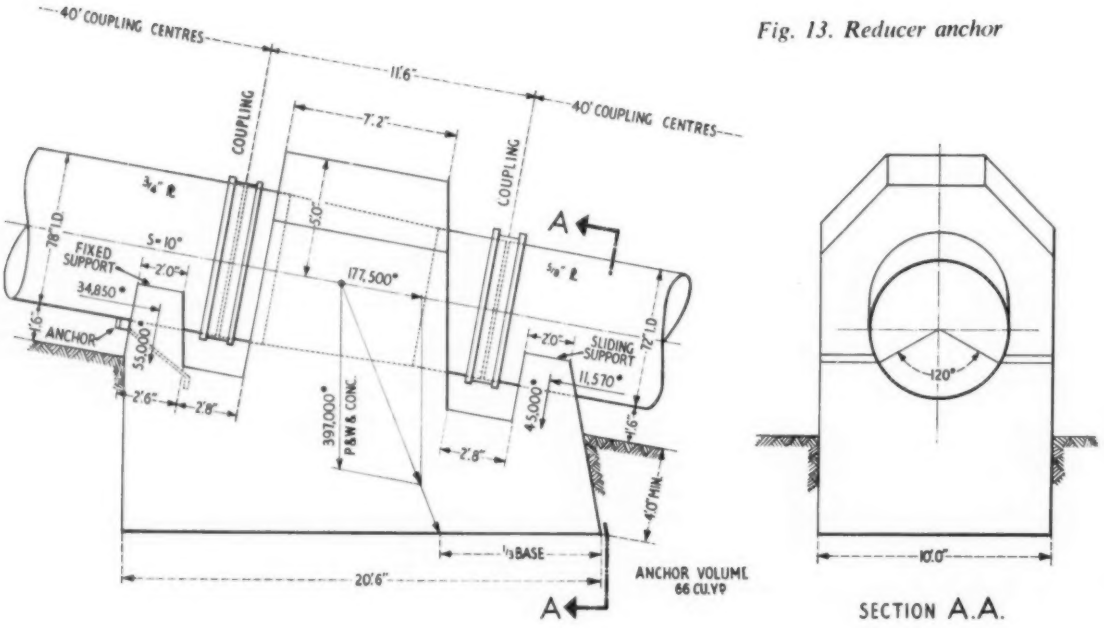


Fig. 13. Reducer anchor



bend with a resultant of 390,000 lb. This will be resisted by the piers, resulting in a lateral force of approximately 43,300 lb. at each pier. Acting in combination with the gravity forces due to the pipe and water load of 97,200 lb. plus the weight of the concrete and earth at 85,000 lb., a resultant force is arrived at which intersects the pier base in its middle third. In the longitudinal direction, the pier was designed for the temperature and gravity forces similar to piers on straight pipe sections. The resistance to temperature movement in the couplings is assumed as identical at each side of a pier and was neglected in the computations. A concrete volume of 18 cu. yards is required for this pier and the load on the soil foundation was determined at 2,100 lb. per sq. ft.

Concrete Anchors. Only three concrete anchors are required for the coupled penstock design, which are located at the upstream end, at the reducer, and at the downstream bend near the powerhouse, as shown in Fig. 7. The anchor at the upstream end is required to facilitate a connection between the concrete transition and the steel pipe. The anchor is extended downstream and provided with a single saddle support for the first 40 ft. pipe section which is coupled to the stub end as shown in Fig. 7, to provide the desired flexibility of the installation.

The reducer anchor shown in Fig. 13 and the anchor near the powerhouse have been combined with saddle piers at each end to carry the adjoining pipe sections coupled to the short central section encased in the anchor concrete. This will provide a flexible installation preventing undue stresses due to settlement of the anchor. The central section of the anchor shown in Fig. 13 was designed for a combination of forces including the unbalanced hydrostatic and dynamic forces, the gravity forces of the empty pipe tending to slide downhill and the resistance to temperature movement in the adjoining couplings. These forces give a net resultant of 177,500 lb. for the expanding condition and a resultant of 159,500 lb. for the contracting condition, along the slope of the penstock. With a horizontal component of 174,800 lb., which is increased to 197,800 lb. by the forces acting on the piers and a sliding coefficient of 0·50 (as used for loose rock and gravel) a dead load of $\frac{197,800}{0{\cdot}50} =$ 395,600 lb. is required on the earth foundation to resist sliding. Deducting the pipe and water load of 130,000 lb. supported on the anchor, a net load of 395,600 – 130,000 = 265,600 lb. and a concrete volume of 65½ cu. yards is required for this anchor. The force on the piers consists of the temperature force of the pipe and water load and the gravity force of the empty pipe. By using a differential coefficient of 0·3, a force of 23,000 lb. was obtained which was added to the above 174,800 lb. horizontal component of the anchor.

Although the forces acting on the anchor at the downstream end near the powerhouse require a concrete volume of only 30 cu. yards, it was necessary to use an anchor of 51 cu. yards to accommodate the two saddle supports for the adjoining penstock sections.

All concrete piers and anchors must be adequately reinforced for the forces acting upon them, especially at the saddle tops of those piers which are subjected to high bending moments tending to crack the concrete at these greatly stressed points.

(To be continued)

Correspondence

The Kariba Flood

To the Editor, WATER POWER

The report on the spring floods on the Zambesi and the accompanying photographs in your June issue are of the greatest interest, and the optimistic note struck at the end of the report is indeed heartening.

The estimation of maximum flood flows is of vital importance both to the engineer designing a dam and to the contractor building it, and it would be helpful to many of your readers if you could publish in some detail the kind of information on the Zambesi floods which had been available to the engineers before the flows of 290,000 and 570,000 cusecs were placed on record.

The point of my inquiry might be most briefly made if I refer to Part Three of the paper on "The Durance Project" in your issue of June 1957. There, reference is made to the difficulty, experienced in some catchments, of fitting observed annual flood peaks into patterns derived from the statistical theory of extreme values. If a table or graph of the annual flood peaks recorded on the Zambesi could be published, it would provide valuable material for a study of general interest.

The Hydrological Branch of the Southern Rhodesian Irrigation Division mentioned in your report, and the consulting engineers, are doubtless engaged on a study of the physical causes of the extreme flood peaks which might occur at Kariba, and it would be of great value to have a picture of the meteorological extremes they would expect, and an account of the meteorological conditions causing the 1957 and 1958 floods.

PETER O. WOLF,
(Reader in Hydrology in the University of London)
Imperial College of Science and Technology, London, S.W.7.

Technical Authorship. The City and Guilds of London Institute, Gresham College, Basinghall Street, London, E.C.2, have issued a pamphlet which contains the regulations and syllabuses for a new scheme of courses and examinations in technical authorship. This course of training has been designed with the needs of the younger entrants in mind, but the examinations will also be open to practising technical authors. Specimen examination papers will be prepared and issued in the autumn to technical colleges interested in setting up courses.

Power Capacitors. Publication No. 401 received from British Insulated Callender's Cables Limited gives full particulars of BICC "MW" power capacitors for 200–600 V operation.

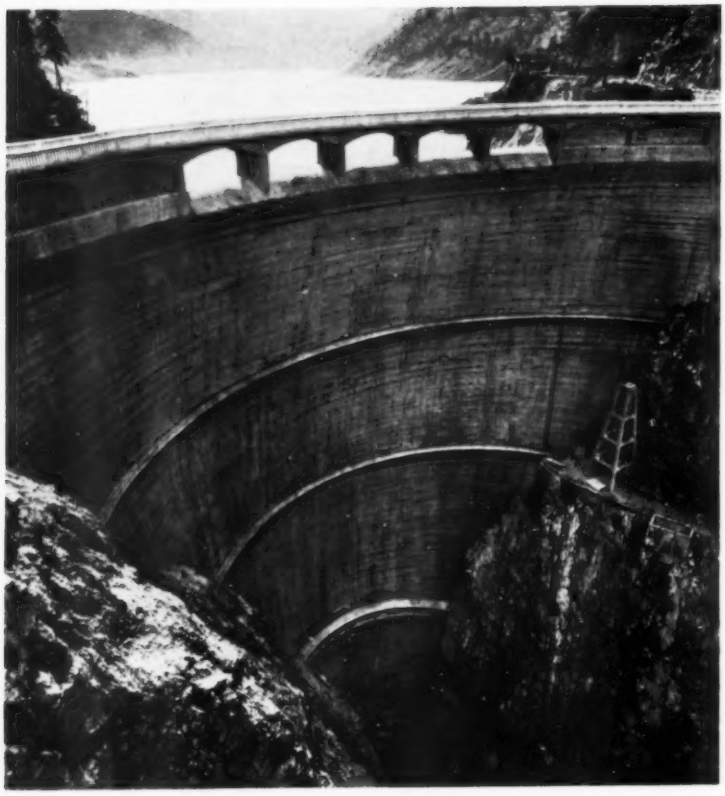


Fig. 45. Lumiei dam seen from downstream

Some SADE Developments

We conclude this series of articles with an account of the Tagliamento scheme. Two interesting dams are to be found on the Lumiei and the Ambiesta, and the station now nearing completion at Somplago will be the largest on the Company's system

PART FIVE

WE have now arrived at the most easterly of the catchments in the SADE territory in which large-scale developments are in course of execution. The Tagliamento rises on the eastern slopes of the Mauria pass and flows east as far as Carnia, where, immediately downstream of the confluence of the Fella, it turns south to emerge into the Friuli plain at Pinzano, at which point the catchment area is 2,300 sq. km.

The power developments on the river—actual and prospective—may be divided into two systems in series, the Upper Tagliamento and the Middle Tagliamento. These are outlined in the map of the catchment area given in Fig. 46. An Eastern system on tributaries of the Tagliamento is also projected, and all these systems will combine to feed an irrigation network based on the Lower Tagliamento.

The Upper Tagliamento system, which is in full service, exploits the levels between 980 m. and 499·35 m. It comprises the main head reservoir of the entire scheme—the Lumiei reservoir, of 70 million cu. m. live storage capacity—and a 60 MW station at Ampezzo. In an average year Ampezzo generates 175 million kWh. The Middle Tagliamento system, which is in course of construction, will utilise the head between 499·35 and 192·9 m. in a 275 MW station at

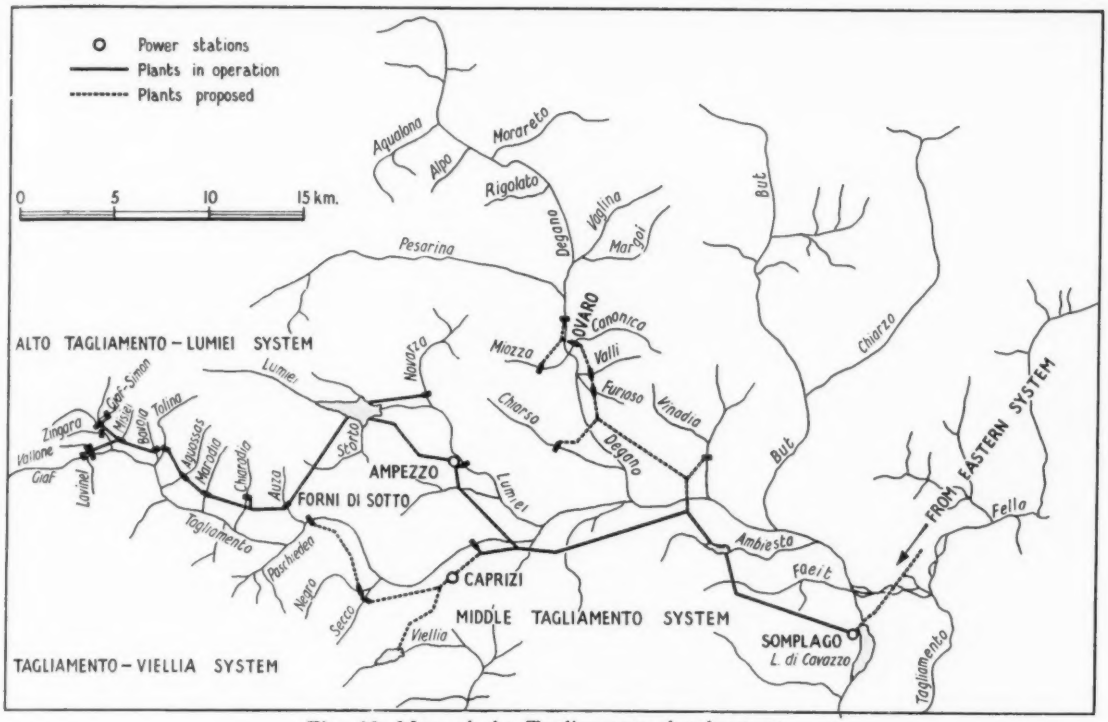


Fig. 46. Map of the Tagliamento development

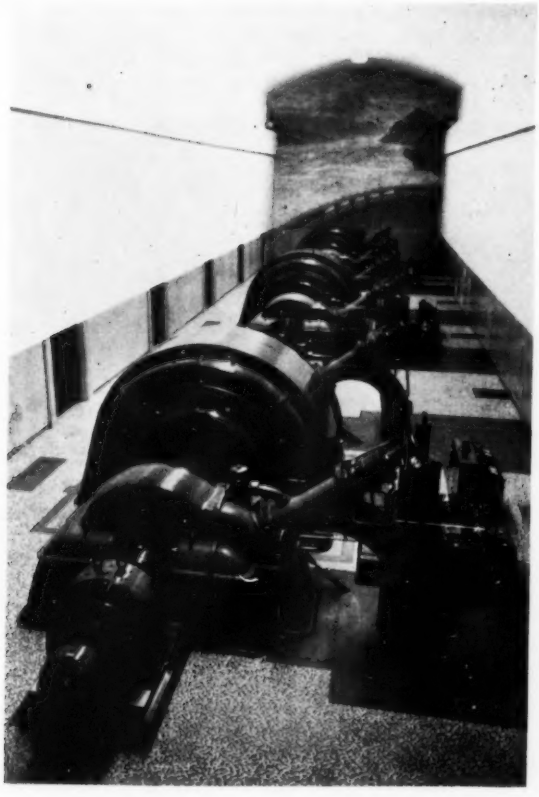


Fig. 47. Interior of Ampezzo station

Somplago, which will be the largest on the SADE network and will be served by a compensating reservoir of 3 million cu. m. capacity on the Ambiesta. This station will have an average annual output of 420 million kWh.

The Eastern system will also most probably supply a second station at Somplago. Altogether it is proposed to install 500 MW of plant in the Tagliamento catchment, and the aggregate average annual production of the Tagliamento will be upwards of 1,500 million kWh.

This will constitute an important contribution to the SADE power resources, not only on account of the amount of power involved but because it represents high-value energy. Ampezzo station, backed by Lumiei reservoir, forms a river-regulation station *in excelsis*, and Somplago, by virtue of its size and of its connection with the daily-service reservoir at Ambiesta, will share with Soverzene the duties of a peak-load and frequency-regulating station for the SADE network.

Upper Tagliamento Scheme—Lumiei Dam and Reservoir

The Lumiei is a left-bank tributary of the Tagliamento, and the dam has been built in the vicinity of Maina di Sauris. The reservoir not only collects the headwaters of the Lumiei but receives the make from a dozen left-bank tributaries of the Tagliamento from its source to Forni di Sotto. To divert these waters it was necessary to construct a free-flow channel, mainly in tunnel, 18 km. long.

A further diversion is that of the Novarza, a left-bank sub-tributary of the Lumiei, on which a small cylindrical arch dam, 38·75 m. high and 9·9 crest

chord, has been erected, and the waters conveyed to the Lumiei reservoir by a 3 km. tunnel. A small tributary of the Novarza—the Veltri—is also dammed at an elevation of 1,296 m. and the waters diverted to Novarza reservoir by a steeply sloping tunnel 124 m. long.

A full account of the design and construction of Lumiei dam is contained in the paper by Dr. Semenza to which we have referred in previous articles, and we must content ourselves with summarising its main features. This dam, which was completed in 1947, was the first of the present series of large double-curvature arch structures to be designed and built by SADE. It is 136·15 m. high, corresponding to a retention level of 980 m., has a crest length of 138 m. and a top chord of 130 m., and contains 100,318 cu. m. of concrete. There is a central spillway 50 m. long consisting of five openings, as seen in Fig. 45, and three tunnel outlets at various levels are also provided. The design is based on the provision of an absolutely symmetrical shell seated within a peripheral joint on a reinforced-concrete saddle, and to secure this symmetry a total of no less than 63,000 cu. m. of rock was excavated. This was justified by the reduction in stress concentrations in the dam, which enabled the dam to be made thinner; the crown thickness is only 16 m. at base and 3·15 m. at the top. It was built by Impresa Torno & Perocco, Milano, the grouting being executed by ICOS.

Ampezzo Power Station

From Lumiei reservoir a pressure tunnel 4,112 m. long and 2·6 m. in diameter has been cut in the right flank of Lumiei valley to the surge chamber of Ampezzo power station. This chamber consists of a vertical shaft, 104 m. deep, with upper and lower expansion chambers, and a single steel penstock, 766 m. long and ranging in diameter from 1·80 to 1·69 m., leads down to the station.

Ampezzo station, the machine hall of which is depicted in Fig. 47, is of the underground type and was completed in 1948. It contains three horizontal double-runner four-jet Pelton sets, each of 20 MW capacity, the turbine having been built by Costruzioni Meccaniche Riva, and the generators by Compagnia Generale di Elettricità, both of Milan. The head on the turbines ranges from 477 to 402 m. As will be noted from Fig. 47, the end wall of the machine hall is embellished with a mural depicting Lumiei dam and reservoir.

Middle Tagliamento System

The Ampezzo tailwaters discharge to an intake works under construction on the Lumiei, where they are joined by river flow entering through a desilting basin, and enter a free-flow tunnel system leading to Ambiesta reservoir. An intake on the Tagliamento near Caprizi also under construction supplements the flow at this point. A northern branch is planned to divert the Degano near Ovaro and pick up the headwaters of its tributaries as well as the flow from the Vinadia. The whole system will comprise tunnels for a length of 40 km.

The Tagliamento is heavily silt laden, and the views given in Figs. 49 and 50, showing the intake works near Caprizi, give some idea of the measures taken to prevent silt entering the tunnel.

A weir across the river (Fig. 49) diverts the waters to an intake works on the right bank, and, from left bank to right, comprises two free overflow spillway openings, each 7·20 m. wide, three Galileo main sluices, each 10·00 m. wide by 4·00 m. high, and two

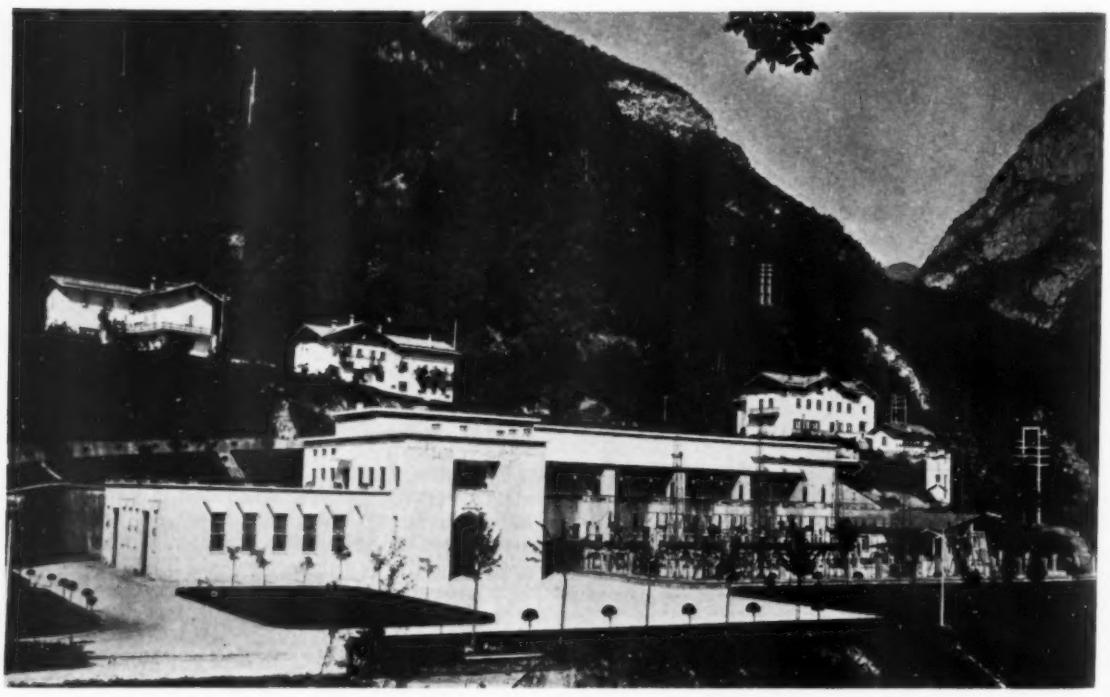


Fig. 48. Ampezzo power station exterior, with employees' houses in the background

Fig. 49. General view of Caprizi intake

Galileo scour sluices, each 6·00 m. wide by 6·00 m. high. These scour sluices remove the coarser shingle which settles in front of the intake screens.

There are two intakes, arranged side by side, leading to the two desilting basins shown under construction in Fig. 50, whence a length of open rectangular concrete-walled channel leads to the intake tunnel.

Ambiesta Dam

The whole of the flow from the tunnel system we have just described discharges to Ambiesta reservoir, which is designed to form a 12 hour storage for Somplago station.

Ambiesta dam is shown in Fig. 53 and an outline of the dam is given in Fig. 52. It is of the double-curvature arch type with a pronounced downstream overhang, and is 60 m. high, 139 m. long at crest and with a span of 120 m. It was built by Imprese Bellanesi Edile Rivrite, Belluno, the grouting being carried out by ICOS.

Particular interest attaches to the design of this dam as it is situated in a potential earthquake zone, and special model tests were conducted at the ISMES Laboratory, Bergamo, on its resistance to earthquake shocks. The region is one characterised by moderate disturbances, with a maximum acceleration horizontally of 0·1*g* and vertically of 0·4*g*. The final model withstood horizontal accelerations of 0·4*g*, acting both in the upstream-downstream direction and along the chord—a value comparing favourably with the maximum of 0·25*g* recorded during the Messina earthquake of 1908. Collapse occurred only after a series of shocks of long duration corresponding to a vertical acceleration of 0·9*g*. Fig. 54 is a photograph of the model after the test to destruction.

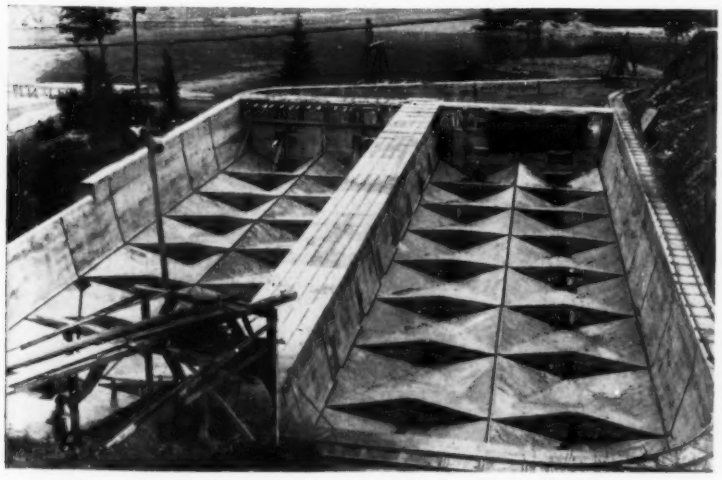


Fig. 50. Caprizi desilting basin under construction

Model tests on the stress system within the structure revealed the

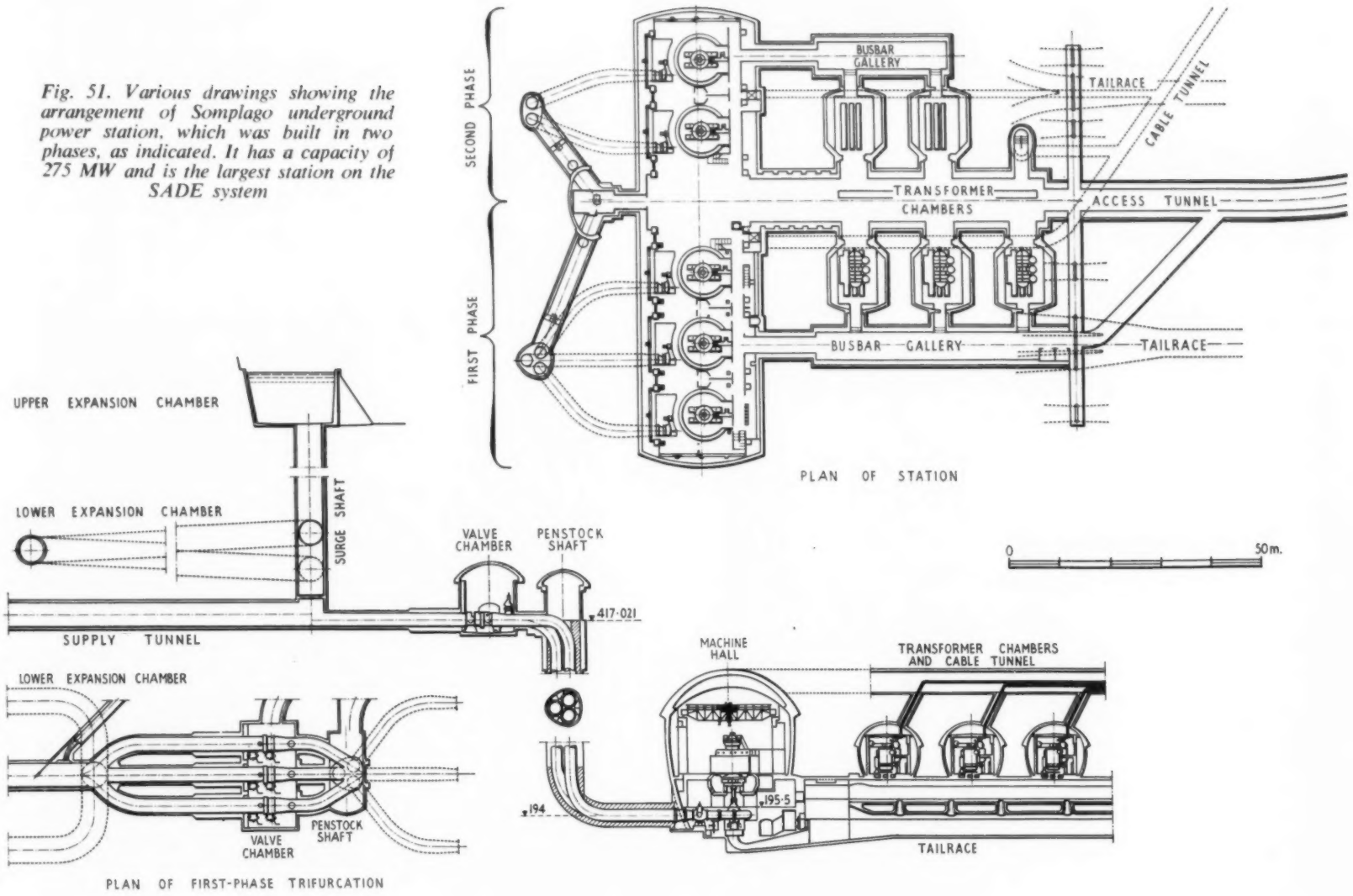


Fig. 51. Various drawings showing the arrangement of Somplago underground power station, which was built in two phases, as indicated. It has a capacity of 275 MW and is the largest station on the SADE system

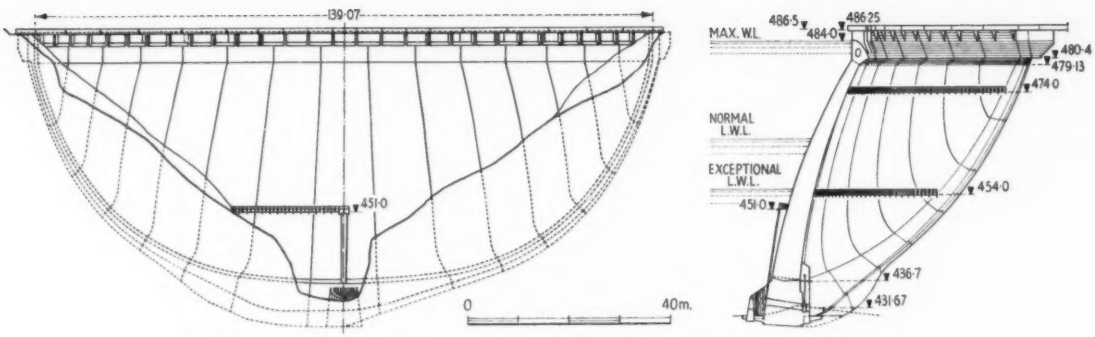


Fig. 52. Upstream elevation and cross-sectional elevation of Ambiesta dam

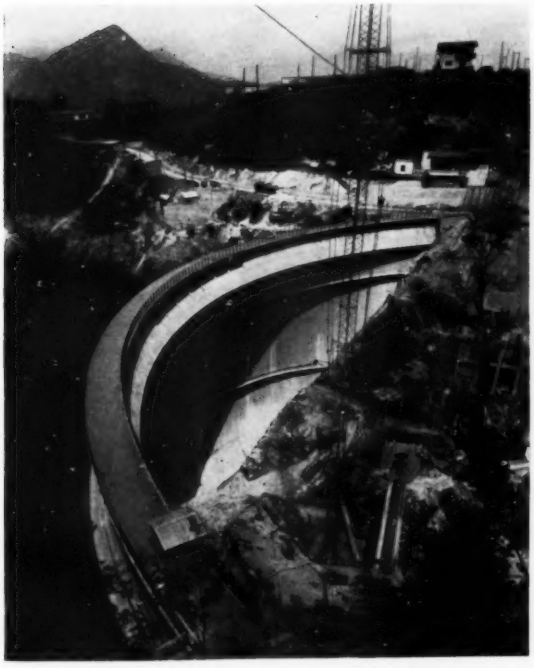


Fig. 53. Ambiesta dam as just completed

necessity for heavily reinforcing the sill of the dam. This enabled the behaviour of the vertical sections as beams encastré at the bottom and freely supported at the top to be taken into account in determining the strength of the dam, and the tests confirmed that this treatment enabled the stresses to be reduced in the ratio of 56 : 33 as compared with a design relying solely on the strength of horizontal arches. To prevent spalling, a light steel net reinforcement was inserted in both upstream and downstream faces, but was not taken into account in the design calculations.

Practical difficulties on the site were presented by a series of fissures, including three major ones, traversing the bed, and the concrete cradle surrounding the peripheral joint had to be heavily reinforced, the average amount of steel introduced being 18 kg. per cu. m. of finished construction. A heavy grouting programme was a natural result of this rock condition. First, shallow injections consolidated the rock to a depth of 10 m., and below this a grout curtain 20—30 m. deep was then put down, the consumption of dry cement per metre length of hole being 10—12 kg. Grouting galleries were also driven in the flanks of the site and are permanently maintained. Excavation for the abutments was taken 10—12 m. into the sides to reach sound rock and to obtain a better design shape for the dam cupola.

The dam is designed for a maximum retention level of 484 m. and has a seven-opening central spillway (the crest is 139 m. long), a high-level discharge tunnel and a ground discharge tunnel.

The intake is situated on the right bank and leads to two parallel tunnels, 9 km. long, feeding respectively the first and second stages of Somplago power station.

Somplago Power Station

Somplago station, as we have said, will be the largest on the SADE network and will eventually contain five 55 MW generating sets. It is now in an advanced stage of construction and is being built in two stages, the first comprising three sets and the second stage the remaining two sets.

The station is of the underground type, and as will be gathered from the drawings, Fig. 51, there are many points of interest in its layout and general

Fig. 54. Earthquake test on model of Ambiesta dam conducted at the ISMES Laboratory, Bergamo

design. Referring to the arrangement of the surge chamber and penstocks for Stage I, shown in the bottom left-hand corner of Fig. 51, the supply tunnel, of 5·15 m. diameter, terminates in a vertical surge shaft 5·25 m. in diameter and 70 m. high. There is a slight constriction at the bottom of this shaft, and immediately above it is a lower expansion chamber of 3,580 cu. m. capacity consisting of a circular-section tunnel forming a rectangular loop. The upper expansion chamber, of 7,363 cu. m. capacity (12,940 cu. m. for both stages) is of the non-overflow type and consists of a reinforced-concrete tank built in the open on the mountainside.

Immediately downstream of the surge shaft the tunnel divides into three parallel branches which lead to a valve chamber, whence three steel penstocks descend through a vertical shaft, 223 m. high and 5·15 m. in equivalent diameter, to the three turbines. In this shaft the penstocks stand free but are clustered in trefoil formation. They are held by a concrete anchor block at the bottom and are yoked every 12 m. Each penstock is 2·55 m. in diameter and is 4 cm. thick at the bottom.

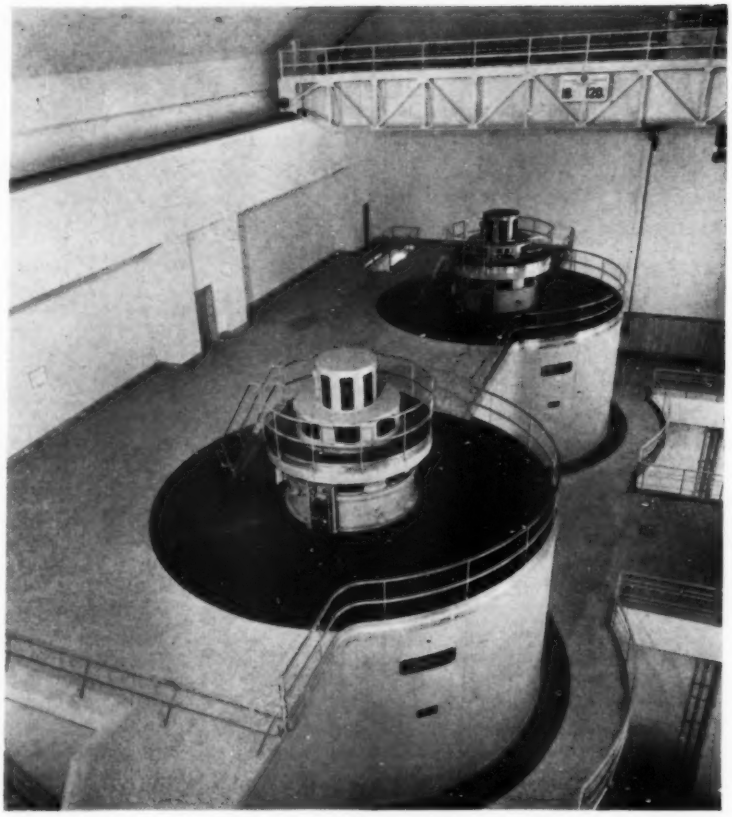


Fig. 55. Two of the generators in Somplago station, Stage I

This use of a common vertical shaft to carry three independent penstocks was found to be the most economical solution of the tunnelling problem, and retained the advantage of an independent head valve for each turbine and permanent access for inspection of the penstocks.

The arrangement for Stage II will be similar, but with all dimensions appropriately reduced to correspond to two generating sets instead of three.

In the view of the machine hall reproduced in Fig. 55, the first two of the three generators constituting Stage I will be seen. These sets, commissioned in 1957, consist of Riva Francis turbines coupled to Marelli generators, running at 428 r.p.m. and operating under a head of 284–225 m. The three machines discharge to a common tailrace tunnel, the first part of which is provided with an upper expansion chamber (see Fig. 51) connected to the main tunnel by a series of orifices, thus acting as a surge chamber. Rock conditions did not favour the excavation of a single enlarged chamber, and the double-chamber arrangement affords the required hydraulic characteristics while simplifying the construction problem.

The influence of rock conditions is also to be seen in the arrangement of the transformers, of which there

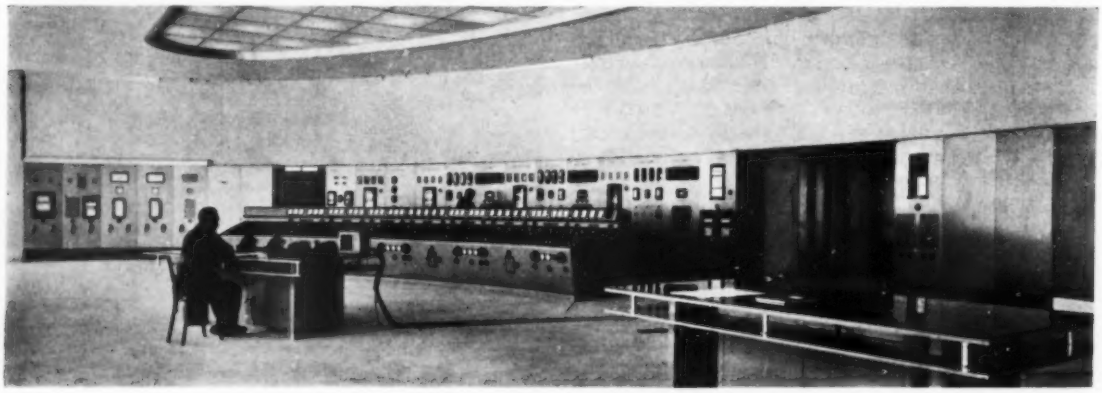


Fig. 56. Somplago station control room

is one 63 MVA 10/138·6/240 kV TIBB three-phase unit for each generator. Each transformer may be either delta or star connected on the high-voltage side to give voltages of 138·6 or 240 kV. The change-over from one to the other connection is made in a few minutes by means of a manually operated device. Each transformer is housed in a separate chamber to one side of the central entrance gallery. A rock pillar 6 m. thick separates adjacent transformer chambers, and the transformers are arranged to be withdrawn endwise so that the span of the chamber roof arch is kept to the minimum. This arrangement also affords the advantage that each transformer is completely isolated for purposes of fire protection.

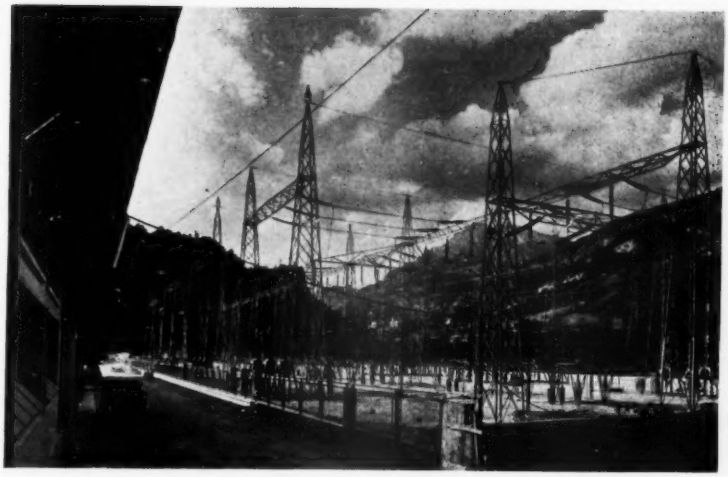


Fig. 57. Somplago outdoor switching station

Connection from the generators is made by aluminium busbars brought along a busbar gallery behind the transformer chambers, and the high-voltage cables rise to a cable tunnel leading to the outdoor switchyard. An interesting detail of this cable tunnel, which is depicted in Fig. 58, is that the cable racks are made of specially shaped tiling.

The control room, Fig. 56, forms the upper storey of the office block, in front of the outdoor switchyard and at the entrance of the cable tunnel. This control-room position has been chosen, not because of excavation difficulties as might be supposed, but for psychological reasons, as the conclusion has been reached that where it is possible to arrange it, the staff are happier in daylight; moreover from the control room it seems to be better to look at the switchyard than at the generator room. Galileo equipment is being installed in the control block and in the switchyard.

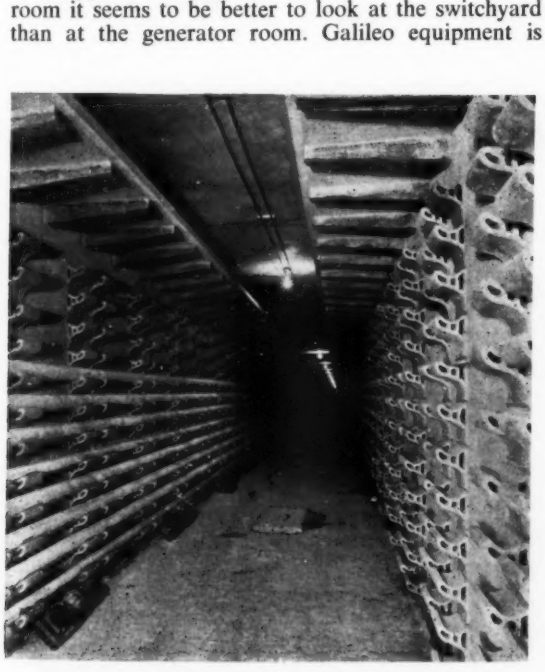


Fig. 58. Ceramic cable racks in Somplago cable tunnel

As a key station in the SADE network, Somplago will form a supply point in the 220 kV grid described in the first article of this series.

The Somplago tailwaters discharge to Lake Cavazzo—a natural lake which forms the weekly regulation reservoir for the irrigation network in the lower part of the catchment. The presence of this lake still further enhances the freedom with which Somplago station can be operated.

Eastern System

An Eastern development is being planned to utilise some left-bank tributaries of the Tagliamento, including the But, Chiarzo and Fella. A diversion from the Gail, in the Austrian Territory, on the north side of the Alpine Main Divide, has been envisaged and is being examined by the interested Companies and the two Governments. The main reservoir, up to 118 million cu. m. capacity, will be at Ramaz, and the waters will supply a station, Somplago II, to be constructed adjacent to the present Somplago I.

Acknowledgments

We are glad to express our indebtedness to Dott. Ing. Carlo Semenza, Manager of the Hydraulic Construction Department of Società Adriatica di Elettricità, for the facilities he so willingly afforded us in connection with the preparation and publication of this series of articles and for the interest he displayed throughout in our work. In Dr. Semenza's unavoidable absence from Italy we were welcomed by Dott. Ing. N.A. Biádene, Vice Director of SADE. All arrangements were most admirably conducted by Dott. Ing. E. B. Gallo, Dr. Semenza's Technical Secretary, and by Dott. Ing. G. Faloma, of the Ufficio Studi; during our tour we received valuable technical information from the whole staff of the Department and especially Dott. Ing. Corrado, Director of the Cellina zone at Pordenone, and from Dott. Ing. L. di Brai, Director of the Tagliamento zone. Ufficio Studi provided us with abundant technical information, drawings and photographs.

C.P. "Tornado" Tunnel Drill

New Consolidated Pneumatic 50 lb. tunnel drill set up for wet drilling

After exhaustive trials with prototype and pre-production models, Consolidated Pneumatic Tool Co. Ltd. have placed on the market a new rock drill which they claim to be the fastest 50 lb. tunnel drill in the world. It is known as the "Tornado" No. 503 Tunnel Drill, and is entirely British designed and built.

At a recent demonstration in a Derbyshire quarry, the drill achieved a penetration rate of 38 in. per min. with a 4 ft. rod and a $1\frac{9}{16}$ in. bit in a hard limestone having an average crushing strength of 29,700 lb. per sq. in. By comparison, a standard CP32F drill—a type equal to a normal 50 lb. class machine—drilled $21\frac{1}{2}$ in. per min. in the same rock face, both drillings being wet.

In the design of the drill a high power-weight ratio has been achieved by using a large-diameter piston (3 in.) combined with a short stroke ($1\frac{7}{8}$ in.), and the air passages and valve are designed to minimise pressure drop and provide immediate acceleration of the piston. A three-position trigger provides for emergency air blowing while the drill is running, normal operation with water, and shutting off. For dustless dry drilling a Hemborn dust extractor can be used in conjunction with a Hemborn drill rod and $1\frac{3}{4}$ in. tungsten-carbide bit. In the tests we have mentioned, SECO tungsten-carbide tipped drilling rods were used.

The standard chuck and retainer are designed for a $\frac{7}{8}$ in. hexagon $\times 4\frac{1}{4}$ in. rod, but alternative fittings can be supplied for 1 in. hexagon rods. The retainer is designed round a bonded rubber ring which acts as a cushion in the event of the collar striking the retainer.

Air was supplied at 90 lb. per sq. in. by a C.P. "Power-Vane" rotary compressor.

Portable Balancing Equipment

Despite the relatively slow speeds involved, the large size of the rotating components of hydro-electric plant presents considerable balancing problems, since dynamic balancing machines of a suitable size are seldom available. This difficulty is overcome by the introduction of the Dawe Type 1254 Portable Balancing Equipment, made by Dawe Instruments Ltd., 99 Uxbridge Road, London, W.5, which enables rotating parts to be balanced *in situ* under their normal running conditions. This new equipment comprises essentially the sensing and measuring circuits of a dynamic balancing machine. In place of the suspension and drive normally incorporated in such a machine, those

Dawe Dynamic Balancing Equipment with pick-up (left) and stroboscope (right)

of the component itself are used. For instance, a water turbine might well be balanced by motoring the associated generator.

In use, the operator applies the vibration pick-up of the equipment to a bearing, and a meter then indicates the amount of vibration arising as a result of unbalance. The position of the unbalance is also indicated by triggering a stroboscopic lamp from the pick-up. From a knowledge of the design, it is now possible by a rather elaborate calculation to determine the correction necessary. To simplify this procedure, however, it is better to add a known mass to the rotor and then to take a second set of readings. By plotting the two sets of readings on the polar diagram provided, the correction required to eliminate unbalance can be read off directly.

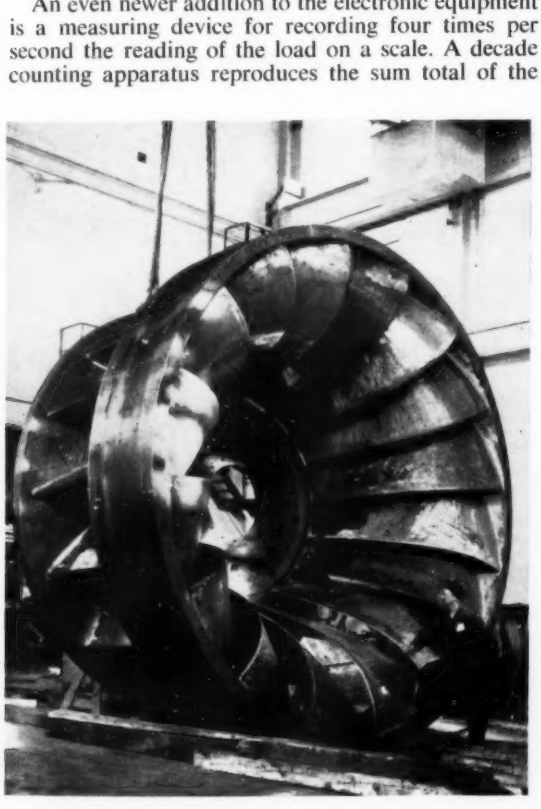
Quite apart from its use for dynamic balancing, the equipment is a very efficient vibration meter. It can be used, for instance, in this way for regular checks to detect whether erosion or deposition of foreign matter, which may result after long service, have affected dynamic balance.

Electronic Testing Equipment for Turbines

Nydqvist & Holm (NOHAB), of Trollhättan, Sweden, have always maintained a keen interest in basic research and recently celebrated the 50th anniversary of their turbine model testing station by having a Press visit to their laboratories. These have now been equipped with a number of electronic measuring devices. Among these may be mentioned a revolution counter provided with a decade counting apparatus which has a counting capacity of 30,000 impulses per second. The impulses are produced by means of a photoelectric pickup from the toothed edge of a rotating disk.

An even newer addition to the electronic equipment is a measuring device for recording four times per second the reading of the load on a scale. A decade counting apparatus reproduces the sum total of the readings and the number of readings. By a simple division it is possible to obtain the arithmetic mean of the load. This apparatus, made by Svenska AB Toledo in collaboration with Svenska AB Philips, is said to be the only one of its kind in use at present. The model testing station utilises a natural head of 31 m. (102 ft.), derived from the Göta river, the efficiencies and outputs being measured on model runners having a standard diameter of 600 mm. (1 ft. 11 in.).

The accompanying illustration shows an outstanding achievement of Nydqvist & Holm A.B., this being one of the three 200,000 h.p. welded runners for the Stornorrfors power station, Sweden. It is 5·2 m. (17 ft. 1 in.) in diameter, is made entirely of stainless steel, and operates under a head of 75 m. (246 ft.).



Welded stainless-steel runner for Stornorrfors power station, Sweden

Ducter Low-Resistance Testing Set

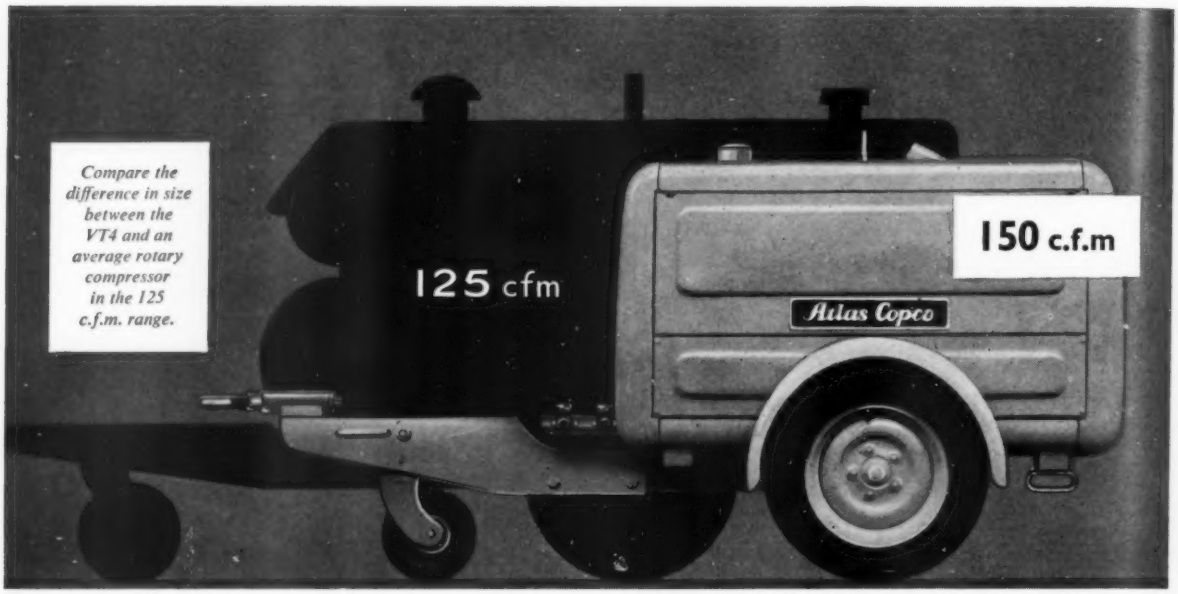
The Ducter Low-Resistance Testing Set or Ducter ohmmeter, manufactured by Evershed and Vignoles Limited, is a direct-reading instrument for measuring low resistances, from a few ohms down to one microhm, with a very precise degree of accuracy. It is especially useful for measuring the resistances of switch contacts, armature windings, transformer windings, copper bars, fuses, soldered joints, earth continuity conductors and various other applications where great accuracy is necessary.

The complete set comprises an ohmmeter, one pair of Duplex handspikes, one pair of single handspikes and an alkaline battery or cell. The ohmmeter consists of a direct reading cross-coil ohmmeter of the type fitted in Megger Insulation Testers. It is contained in a strong hardwood case, provided with carrying handle, range switch, cut-out and resetting switch, two terminals at one end for the battery and four at the other for the current and potential connections. The scale length is 4·5/16 in., each instrument being individually calibrated.

The movement is of the permanent-magnet moving-coil type. The moving element consists of two coils rigidly secured at an angle to one another and pivoted in spring-mounted jewels so that the coils swing in the annular gap between a fixed centre iron and the polar faces of a permanent magnet. The centre coil is connected across a shunt in the main circuit and the current in this coil is therefore proportional to the

NO OTHER COMPRESSOR

—piston or rotary—delivers so much air for its size and weight


Compare the difference in size between the VT4 and an average rotary compressor in the 125 c.f.m. range.
125 cfm
150 c.f.m
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Lighter and smaller than some machines with a free air delivery of only 60 c.f.m.—the VT4 has taken power weight ratios into a new dimension.

Powered by a Deutz diesel engine the VT4 gives an unequalled c.f.m./h.p. ratio, with fuel economies which offer a saving on any contract.

The VT4 is the first of a series based on major changes in design and production methods. For example, it is the only machine with a single welded unit, which incorporates the crankcase, bellhousing and cylinder ducting.

Basic simplicity in design eliminates maintenance and overhaul problems—local mechanics anywhere can service this machine.

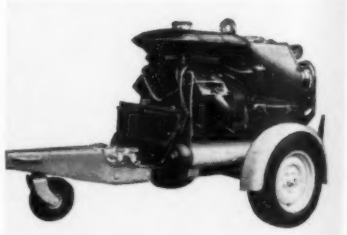
Unscrew one nut and all four valves in the low pressure cylinder can be removed. Loosen three bolts and the complete engine-compressor unit lifts out.

The VT4 is 100% air-cooled—starts on the button and works with equal efficiency anywhere in the world.

The VT4 is easier to load with its low weight and built-in lifting eye—easier to tow with its two wheels and torsion bar suspension.

Manhandling on site is a simple operation with a machine weighing less than 21 cwt. and fitted with a pivoting nose wheel.

All instruments and controls are operated from a central dashboard.



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P.26

current in the resistance under test. The deflecting coil is connected across the resistance under test, and carries a current which is proportional to the potential drop.

The polarity of the two coils is so arranged that the torques developed oppose one another. The position of the movement is therefore dependent on the ratio between the potential drop and the current, i.e. (by Ohm's law) on the value of the resistance being measured. The scale is calibrated directly in ohms or microhms. As the instrument is a true ohmmeter, its accuracy is unaffected by variation in battery voltage, and no adjustment is required, the instrument being always ready for use.

Abstracts from the World Technical Press

Buckling in Thin-Walled Pipes

This study deals especially with the steel linings of power shafts and tunnels. A pipe intended to operate under pressure must be designed so as to resist the internal pressures to which it will be submitted during its service life without being damaged and with an adequate margin of safety. On the other hand, especially in the case of steel linings in shafts and tunnels, the pipe, when empty, is submitted to external pressures which are the more dangerous as it is uncertain how they arise. The author makes a study of the resistance of such linings to external pressure, that is to say to buckling, beginning with a survey of damage to linings due to the back-pressure caused by seepage or grouting. He then deals with buckling proper in relation to the instability of the wall and the elastic limits of steel. After applying this mathematical study to shaft and tunnel linings, he describes the laboratory tests carried out to check the results of his theory, as well as the testing equipment used. In its conclusions, the author strongly advocates the use in such lining of steel with a high elastic limit, and stresses the usefulness of large-scale field tests and measurements effected on actual linings on which adequately precise data are available. (H. Borot, Société Dauphinoise d'Etudes et de Montages, *La Houille Blanche*, Vol. 12, No. 6, December 1957, p. 881, 7 pp., 8 ff.)

Power Conduits in High-Head Developments

Under the general term of power conduits the author comprises all the water-supply works between the storage reservoir and the power house, as well as between the latter and the tailwater. He devotes his main attention to underground conduits consisting either exclusively of pressure tunnels, or of pressure shafts, vertical or inclined, connecting with pressure tunnels. Of the five layouts reviewed, only one has an outdoor power house, the turbines of which discharge into an open tailrace. The only difference between this layout and the first variant is that the latter has an underground power house sited fairly close to the foot of the hill and connected with the tailwater through a short discharge tunnel. In all other three patterns, the underground power house is far away from the foot of the hill, power conduits varying from inclined to vertical shaft or to a combination of both, the main common features being an underground surge-tank and a long tailrace tunnel. The two layouts, including an inclined shaft, also comprise an access tunnel and an aeration shaft, whereas the plant fed by a vertical power shaft has also a vertical access-and-aeration shaft. The alternatives of a pipeline housed in a tunnel and of two power tunnels are also examined. The various types of tunnel are then discussed—unlined, concrete-lined without reinforcement or any additional measures, steel-lined, etc. Stress is laid on the importance of inner steel shells with concrete packing between shell and tunnel wall. In instances where an adequate overlying stratum is available, chequered foil, either copper, "Opanol" or similar, inserted between the outer and the inner concrete lining, can ensure the watertightness of the tunnel. The foil is glued to the outer lining, and subsequently covered by the inner lining. Application of this method, however, is restricted to tunnels where the surrounding rock is assumed to be pervious, so that it could be penetrated by water leaking from the tunnel. The problem of power conduits is finally considered from the economic aspect, and the results of these considerations are condensed in a table for twelve various layouts with either reinforced or prestressed concrete linings, each layout being accompanied by a small schematic diagram. (Hermann Berger, *Die Wasserwirtschaft*, Vol. 48, No. 5, February 1958, p. 121, 5 pp.; No. 6, March 1958, p. 137, 5 pp., 25 ff.)

A New Pipe-Laying Rig

This self-propelled rig, which was developed by American Pipe & Construction Co. for a Colorado River aqueduct contract, on which it is now in operation, might well find a place in the not too distant future in the construction equipment for hydro-electric schemes. The Pipemobile, as it is called, is a machine powered by a 225 h.p. diesel engine driving a 250 kVA d.c. generator, that can insert itself into a pipe section, pick it up, move it from a trench-side storage area into position, and seat the pipe joint. To enable it to enter and leave the pipe without damaging the steel joint ring, the Pipemobile has two sets of front wheels arranged in tandem. As the rig enters a pipe section, there is no load on the first of the wheels. Load is taken by this front wheel as the second set of front wheels is retracted and moved into the pipe. Load is then transferred to the second set of wheels, and the process is repeated as the front end of the Pipemobile passes out of the pipe. The front wheel is single rubber-tyred as it is used only for entering and leaving the pipe, whereas the second set of front wheels, which moves the pipe, thus needing greater control and dealing with heavier load, is double-tyred. There are two two-wheel sets on the back end of the Pipemobile to ensure stability during travelling and to support the heavy machinery load on the rear end. Each set of wheels is driven by a 50 h.p. electric motor connected to the wheels through a gear reducer and

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1. *Boulder blasting with the BBD 11 LH.*
2. *Plug hole drilling in concrete.*
3. *Scaling with the BBD 11 LTS. Note the convenient leg support.*
4. *Survey hole drilling using the BMK 11 pneumatic pusher.*

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chain drive. Maximum speed of the rig is 2½ miles per hour; speed of travel is adjusted by changing the output voltage of the main generator. The machine is steered by an operator who rides on the front end and controls the front wheel, while the main operator is located at the rear end of the machine, both operators being connected by an intercom system to ensure perfect co-ordination of manoeuvring, especially when negotiating turns. If required, any one of the sets of wheels can be disconnected for turning purposes. The Pipemobile has hydraulically operated horizontal struts that exert pressure against the sides of the pipe to keep the lifting load distributed. After these horizontal struts are firmly in position, vertical hydraulic rams lift the pipe off the ground. These horizontal and vertical rams also permit close adjustment in seating the pipe joint. On the job instanced, the pipes are 13 and 13½ ft. in diameter; they are laid in 16 ft. sections, weighing 63 or 68 tons. When laying pipe sections on steep slopes (a photographic illustration shows the rig in operation on a 29 deg. slope), the extra push required is given by tractors; on some steeper sections, the machine lays the pipe downhill instead of up. On steep slopes, the pipe sections are anchored by a steel angle welded to a plate held in a previously cast concrete block, the angle being welded after pipe has been seated. Laying procedure and manufacture of pipes are described in detail. (*Engineering News-Record*, Vol. 160, No. 2, January 9, 1958, p. 23, 5 pp., 7 ff.)

Methods of Calculation for Arch Dams

This article by the engineer in charge of dam design at the National Civil Engineering Laboratory of Portugal is a comprehensive and critical review of the more important methods of calculation employed in the design of arch dams. Each method is concisely explained and its merits and limitations are lucidly pointed out. The author commences with a brief historical sketch in which he refers to the earlier use of the pipe formula and surveys the various methods that have been proposed. He then presents a simple diagram showing the complete system of forces and moments acting upon an element of a dam, and with the aid of this diagram proceeds to examine various methods of analysis.

In the first instance he traces the development of the method of independent arches, but in spite of the various refinements introduced by successive workers he considers that the main objection to this method is that it takes no account of the vertical continuity of the dam. In the case of thin dams of high curvature constructed in deep narrow valleys satisfactory results can be obtained for the central zone of the dam, but it takes no account of the effects of the shape of the valley, of encastrement, or of the crest parapet on the behaviour of the dam. In the case of thick dams the method can result in dangerous tensile stresses in the lower aches, and to counteract this the method of active arches was devised. The method of inclined arches, next considered, introduces some vertical action at the expense of the arching effect. It cannot take into account the effect of any asymmetry in the design of the dam, and although it gives a first approximation for the values of the horizontal transverse forces and moments it is far from giving an adequate analysis of the general system of stresses in a dam.

Methods based on shell theory are next considered. These regard a dam as a surface of revolution and derive expressions for the equilibrium of a point on this surface. Obvious criticisms of such methods are that the thickness of the dam in relation to the radius of curvature is usually too great to allow it to be neglected, the thicknesses of horizontal sections are normally not uniform, and the centres of curvature of the upstream and downstream faces are never on the same vertical.

A discussion of methods based on an interlacing system of horizontal arches and vertical cantilevers naturally leads to a consideration of the trial-load method. In principle, the full application of this method can give a full analysis of the stress system in a dam, but the overriding objection is the time needed for the calculations; in the case of a dam that is asymmetrical, has an irregular foundation, or the design of which is based on curves other than simple circular arcs, the work involved becomes prohibitive. Investigations in Portugal have shown conclusively that design by means of model tests is quicker and more precise than a full trial-load analysis, and the procedure is now followed of using a simplified trial-load analysis to determine a preliminary design which is then finalised by model tests. Some interesting conclusions have been drawn from a comparison between calculated and experimental results. In the case of symmetrical dams in valleys of regular profile the stresses in the arches derived from trial-load calculations are identical with those obtained experimentally, whereas the calculated stresses at the base of the cantilevers are 20–30% too high. The radial deflections given by the simplified calculations are exaggerated, and near the abutments in the upper part of the dam can be seriously in error in percentage and even in sign. Further, the forces exerted on the abutments by the upper arches as given by calculation can be far in excess of the actual values, particularly where the valley is in the shape of an open V.

Finally, reference is made to the English relaxation method, which is regarded as a contribution to arch-dam analysis of the utmost importance, as it not only gives a most rigorous solution of the calculation of an arch dam but also provides a practical means of solving any problem of tridimensional elasticity. The main objections to this method are the complexity of the calculations, which require the services of specialists, the long delay in obtaining results, and the difficulty of applying the method to the modern profiles used in arch-dam design. (Joaquim Laginha Serafim, *Electricidade*, No. 5, January–March 1958, p. 35, 17 pp., 9 ff.)

Schärding-Neuhaus Inn Project

The Schärding-Neuhaus power station is planned as the fourth and lowest stage of the Lower Inn development and will operate by regulated releases in co-ordination with the Braunau, Erig and Obernberg plants. Natural conditions at the construction site are particularly favourable to the setting up of a safe backwater area. There is a steep high terrace on the Austrian side (right-bank) and an uninhabited flood plain on the Bavarian side, where the works will connect with the 16 km. dyke extending right to the Obenberg plant. The major part of the construction work, including the generating block, the control building and the outdoor switchyard, is planned on

the Austrian bank which is not only safe from flood dangers but also enjoys the benefit of an excellent road and railway network. On the left bank, the foundation pit of the spillway block will be protected by an earthbank enclosing a storage yard and connecting upstream with the Obernberg dyke. The low-deck type of power house, which has proved so satisfactory in eight other plants on the river Inn, has also been adopted for Schärding. One of the main operational advantages of this design is that the main service cranes can travel from bank to bank as the roof of the power-house block and the spillway bridge are at the same level. The turbine pits are covered by sliding lids. As at Braunau, the flow of the turbine inlets is at the same level as the spillway floor. Altogether the Schräding project has been worked out on very much the same lines as the Braunau plant. A guide wall separates the spillway from the power-house block, to ensure an undisturbed flow of the discharge from the turbines. There are four 31,800 h.p. Kaplan turbines, each with a discharge capacity of 245 cu. m. per sec. under a head of 11 m. and driving 30,000 kVA generators at 83·33 r.p.m. The average yearly production, with the plant operating in part as normal run-of-river unit, is 544·67 million kWh, of which 35·5% can be classified as winter energy. Upon completion, the Schärding-Neuhaus plant is to act as a balancing unit to the Lower Inn chain of plants which will have an aggregate capacity of 340 MW and a yearly output of 2,000 million kWh. (Dipl.-Ing. Johann Zelenka, *Osterreichische Zeitschrift für Elektrizitätswirtschaft*, Vol. 10, No. 11, November 1957, p. 403, 10 pp., 8 ff.)

Desanding Arrangements

In this paper, read before the Technical Committee of the Société Hydrotechnique de France on June 28, 1957, the author deals exclusively with the problem of sand trapping at the intake, that is to say the elimination of particles varying in size between a few hundredths of a mm. and a few mm., to the exclusion of silt (particles below the sand-particle size) and gravel (size above sand particles). Desanding installations are divided into two groups—settling tanks, in which settling and removal of the suspended material are effected in two successive operations, and sandtraps or desanders proper, performing both operations simultaneously. Since the use of settling tanks is restricted to small and medium-capacity intakes, emphasis is laid on sand traps, either of the slow-flowing or fast-flowing types, the flushing device being spread over the whole length of the structure in the former (Dufour Desander No. 2, Pont-Escoffier and Pralognan plants), and at the downstream end in the latter (Dufour No. 3, frontal flushing; Montagné, bottom flushing; and Lévin, side flushing). In the second part of the paper, the author analyses, on the basis of the theory of transport and settling, the settling of sand in running water at various flow velocities, and ends his study with a survey of a few problems arising in actual practice in modern installations, this survey being divided into two sections: operational problems, and equipment. These problems include the economic aspects of continuous desanding in intakes provided with an abundance of equipment, especially in the case of water originating from glaciers and containing very fine particles; optimum mode of operating intakes at which strong concentrations of solid transport are restricted to short periods; finally, sand trapping in multi-intake developments drawing their water supply from catchments widely differing in nature. The series of measurements taken in recent years in modern plants should prove useful as a starting point for further investigations. The importance of the problem is perhaps best illustrated by the figure of 10 million kWh, given in the course of the discussion, for the loss of output caused by desanding at the Passy plant (Haute-Savoie) in 1955. (R. Maitre, Ingénieur, EDF Hydro-Production Division, *La Houille Blanche*, Vol. XIII, January–February 1958, No. 1, p. 25, 15 pp., 15 ff.)

Nagarjunasagar Project

Based on the Report of the Project for its first year 1956–57, this article gives a detailed description of this proposal which, compared with other river-valley developments in India, will be a huge undertaking. Its ultimate storage capacity of 9·3 million acre-ft. exceeds the Bhakra capacity by 1·9 million acre-ft. The masonry gravity dam to be erected near Nandikonda village across the Krishna river will be 4,720 ft. in length, including a 1,880 ft. spillway section in the middle. The non-spillway and power blocks will reach a height of about 350 ft. above foundation level. The main power house on the left bank is to have a capacity of 300,000 kW, and a secondary power plant set up at the head of the righ-bank irrigation canal will generate seasonal power to the extent of 60,000 kW. (*Indian Journal of Power and River Valley Development*, Vol. VIII, No. 1, January 1958, p. 5, 6 pp.)

CLASSIFIED ADVERTISEMENTS

Announcements for this column can be accepted up to the 8th of the month for the following month's Issue. The charges are fourpence per word with a minimum of 10s. Box No. facilities 2s. 6d. extra. In order to avoid accountancy it would be appreciated if instructions to insert were accompanied with remittance.

Displayed classified advertising for situations vacant, situations wanted, public appointments, are accepted at £2 10s. 0d. per column inch.

Auckland's Golden Jubilee Electrical Exhibition
Auckland, New Zealand—Easter 1959

THE AUCKLAND ELECTRIC-POWER BOARD is staging an Electrical Exhibition to be held in conjunction with the Manufacturers' Easter Show, at the Epsom Showgrounds, Auckland, for three weeks extending over the Easter period of 1959. (Attendance 1958—245,509.)

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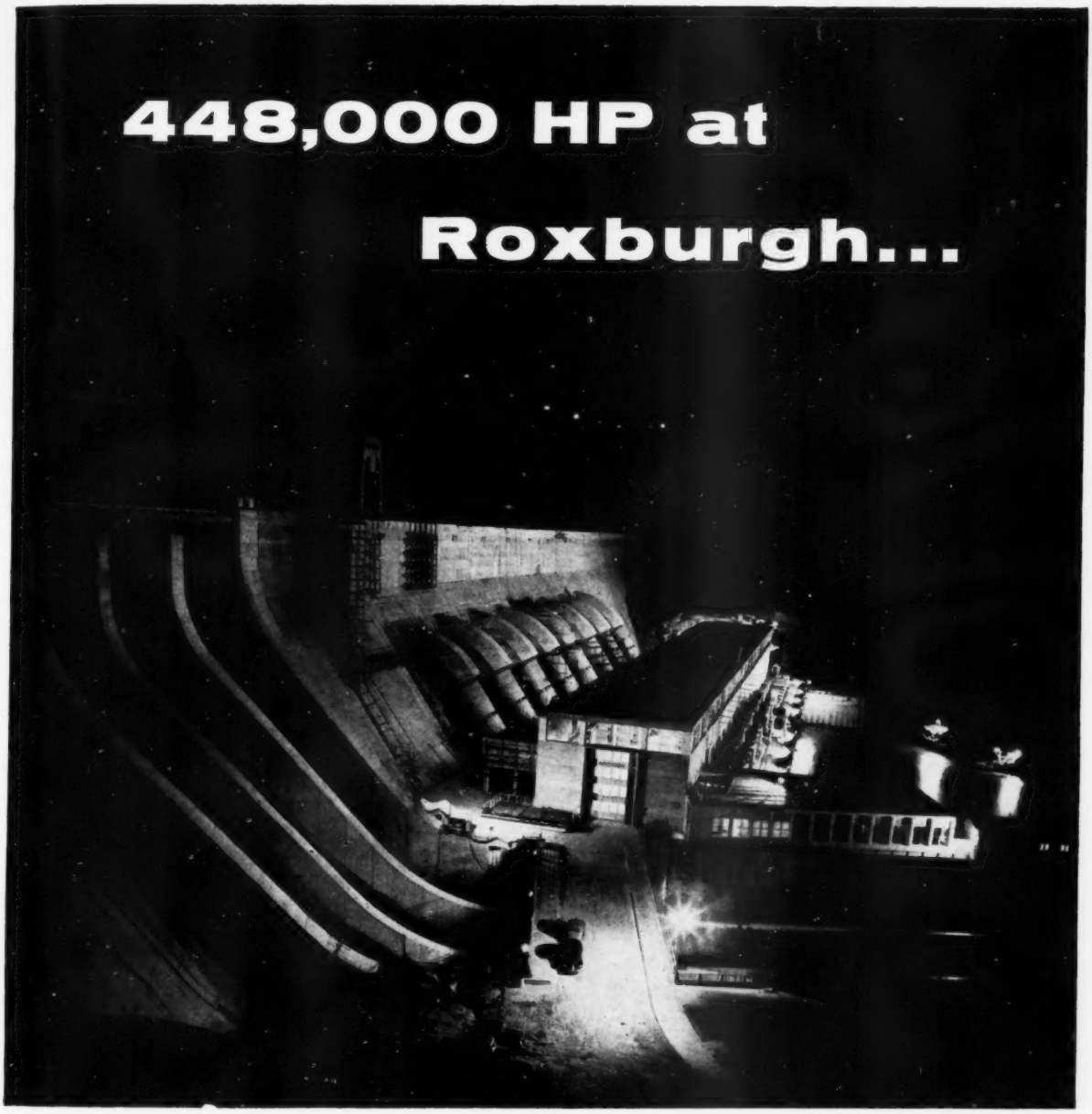

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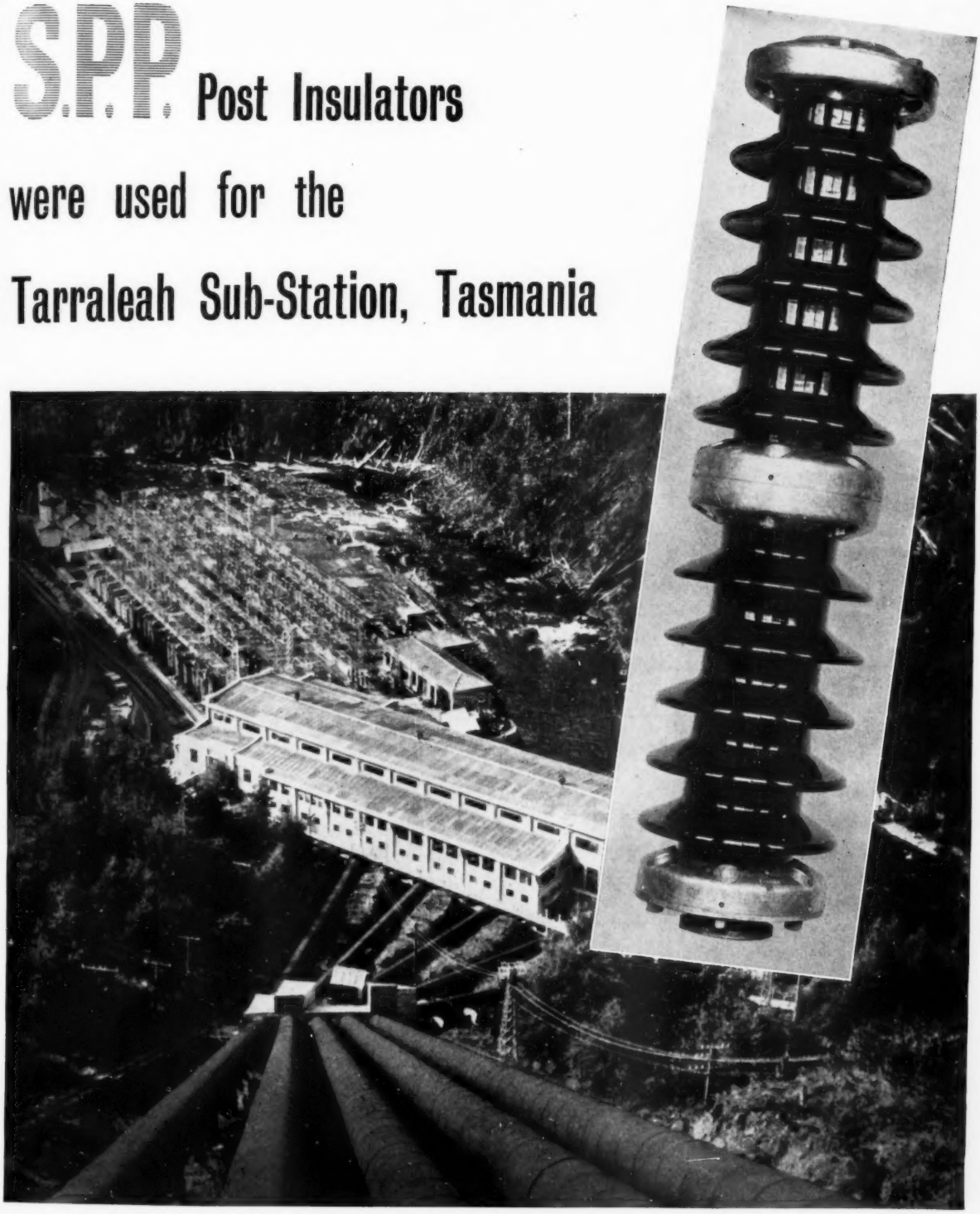

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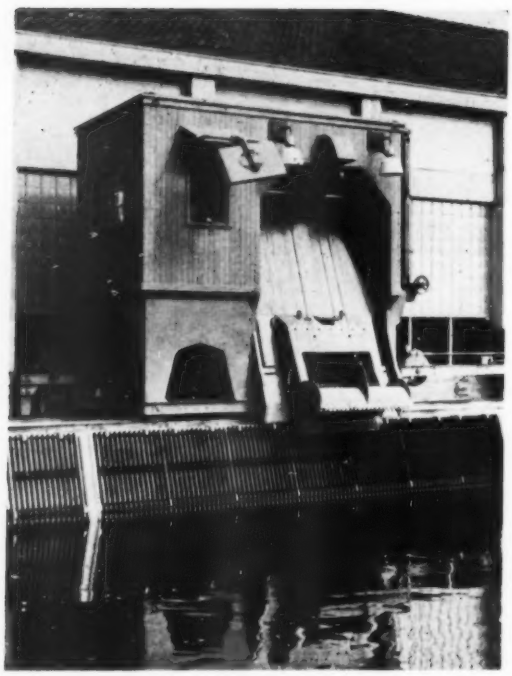

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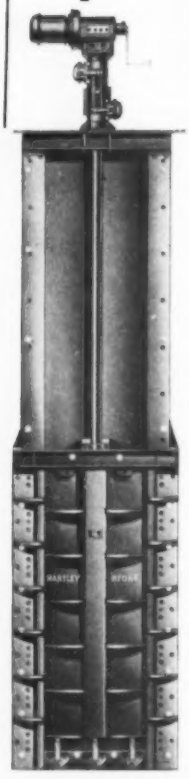

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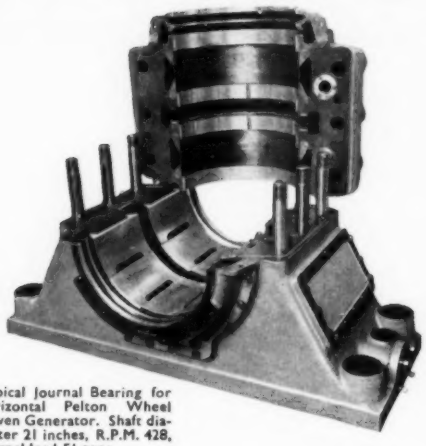

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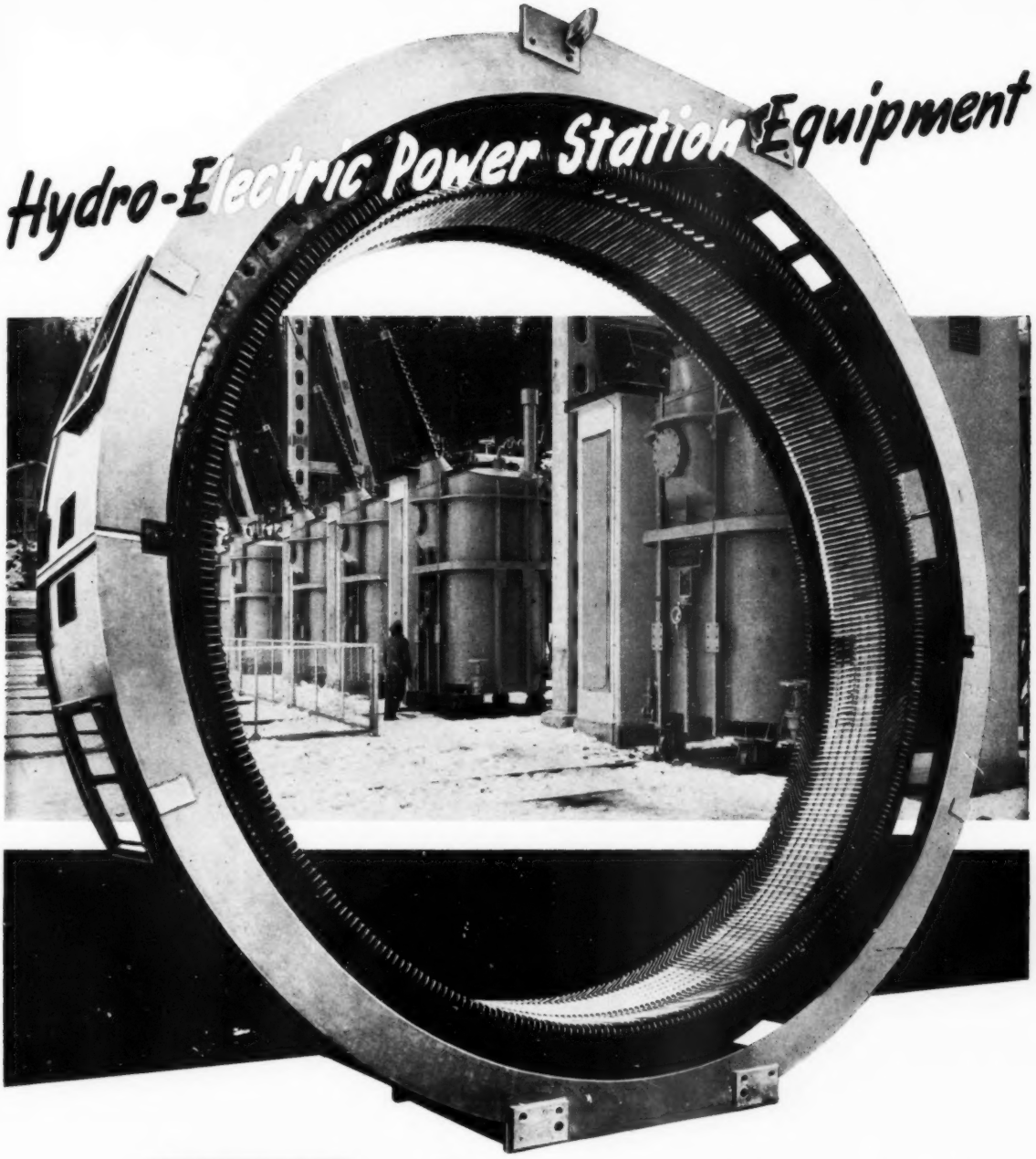

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